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The Effect of Storage on Storm and Combined Sewers

Research Report No. 34



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Research Program for the Abatement of Municipal Pollution
under Provisions of the Canada-Ontario Agreement
on Great Lakes Water Quality

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The Effect of Storage on Storm and Combined Sewers

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These RESEARCH REPORTS describe the results of investigations funded under the Research Program for the Abatement of Municipal Pollution within the Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality. They provide a central source of information on the studies carried out in this program through in-house projects by both Environment Canada and the Ontario Ministry of the Environment, and contracts with municipalities, research institutions and industrial organizations.

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THE EFFECT OF STORAGE
ON STORM AND COMBINED SEWERS

POLLUTION CONTROL
BRANCH

MAR 23 1976

MINISTRY OF THE
ENVIRONMENT

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RESEARCH PROGRAM FOR THE ABATEMENT
OF MUNICIPAL POLLUTION WITHIN THE
PROVISIONS OF THE CANADA-ONTARIO
AGREEMENT ON GREAT LAKES WATER QUALITY

PROJECT NO. 74-8-1

February, 1976

This document may be obtained from -

Training and Technology Transfer
Division (Water)
Environmental Protection Service
Environment Canada
Ottawa, Ontario
K1A 0H3

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ABSTRACT

The effect of storage on storm and combined sewers has been investigated for a residential subdivision of approximately 100 acres area, under development in Southern Ontario. The storage methods examined were:

- (1) ponding of rainfall on flat roofs;
- (2) on-site storage of roof runoff;
- (3) storage of flows from street gutters at catch basins; and
- (4) holding reservoirs as part of the sewer network.

A hydrograph model for estimating stormwater runoff from residential areas has been developed and incorporated in a computer program. The model was used to determine the outflow hydrograph at the storm sewer outfall from the subdivision for a synthetic two-year design storm.

An alternative design, replacing the separate sanitary and storm sewers by combined sewers, was carried out and the outflow hydrograph from a combined sewer network was determined.

The effects of different storage methods on the outflow hydrograph were tested for both the separate and combined systems. Substantial reductions in peak flow were found to occur as the level of storage was increased, with holding reservoirs providing the greatest reductions.

Cost data from the original separate system design were used to calibrate formulae for estimating sewer pipe costs. The total costs of systems incorporating various storage options were compared. A combined sewer system was found to be slightly less expensive than the separate system and small savings in subdivision sewer costs resulted when storage was incorporated.

RÉSUMÉ

Il arrive que des eaux s'accumulent en certains endroits, et les effets de telles accumulations sur les réseaux d'égout pluviaux et unitaires d'une zone résidentielle d'environ 100 acres, que l'on aménage présentement dans le sud de l'Ontario ont été étudiés. Les travaux ont porté sur:

- 1) les eaux de pluie retenues sur les toitures plates;
- 2) les eaux tombées des toitures et retenues sur place;
- 3) les eaux provenant de caniveaux et de bassins collecteurs et retenues en certains endroits; et
- 4) les eaux retenues dans des bassins faisant partie des réseaux.

Un modèle des régimes permettant d'estimer l'écoulement des eaux de pluie des zones résidentielles a été mis au point et incorporé à un programme d'ordinateur. Le modèle a été utilisé pour déterminer le régime de la décharge du réseau d'égout pluvial de la zone pour une période synthétique de deux ans.

Un autre système, où les réseaux sanitaires et pluviaux sont remplacés par un système unitaire, a aussi été élaboré, et on en a déterminé le régime à la décharge.

Les effets de types différents d'accumulations sur le régime des décharges ont été étudiés pour les réseaux séparés et unitaires. On a trouvé que l'importance de la réduction des débits maximums était fonction de celle de l'accumulation, les bassins permettant d'obtenir les plus fortes réductions.

Les coûts du réseau séparé initial ont été utilisés pour calibrer la formule permettant d'évaluer les coûts des tuyaux d'égout. Les coûts totaux des réseaux comprenant divers types d'accumulation ont été comparés. On a trouvé que le réseau unitaire serait légèrement moins cher que les réseaux séparés et que l'utilisation de l'accumulation permettrait de réduire légèrement les coûts du réseau de cette zone résidentielle.

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CONCLUSIONS

The following conclusions were drawn as a result of the study:

1. The cost of combined sewers for the 101.6 acre subdivision studied in this research was estimated to be only slightly less than the total cost of sanitary and storm sewers. If higher full-flow velocities are required for combined sewers than for separate storm sewers, the use of combined sewers in new developments may not be justified. However, choice of the wastewater collection system for any area will be governed ultimately by factors particular to that area, such as topography.
2. The savings in sewer costs resulting from the incorporation of storage in the subdivision were found to be quite small for both storm and combined sewers. Reduction in volumes of untreated overflows to receiving waters is a further benefit when a combined sewer system with storage is utilized.
3. Roof ponding to eliminate roof runoff was found to reduce peak sewer flows from the subdivision by up to 30 per cent and catch basin storage provided similar reductions. Because of different flows to each of the catch basins, storage volumes for a constant outflow varied from zero up to a maximum of 1,500 cu ft.
4. Holding reservoirs proved to be the most effective means of modifying stormwater flows from the subdivision. A graphical relationship between storage volume and peak flow was developed for a holding reservoir, enabling storage requirements for any level of peak flow reduction to be estimated.
5. The rational method, which considers only average rainfall intensity for a selected rainfall duration, is not suitable for design of sewer systems incorporating storage. A runoff model which computes complete sewer flow hydrographs from a design

storm was developed and when tested, the model produced results consistent with field data from other sources.

6. The model was used to predict the runoff hydrograph from the subdivision for a two-year design storm. The contributions of different land uses to storm sewer flows were:

<u>Land Use</u>	<u>Area (acres)</u>	<u>Peak Flow (cfs)</u>	<u>Peak Flow/ Unit Area (cfs/acre)</u>	<u>Stormwater Volume (cu ft)</u>
Residential	83.2	94.7	1.14	202,500
Industrial, School	10.5	21.2	2.02	50,000
<u>Park</u>	<u>7.9</u>	<u>5.0</u>	<u>0.63</u>	<u>11,900</u>
Total	101.6	120.9	1.19	264,400

Although they occupy only a small part of the catchment, the industrial areas and the school make a relatively large contribution to stormwater flows because of their high imperviousness. The higher cfs/acre value suggests that the benefits of storage in these areas should be investigated.

RECOMMENDATIONS

1. It is recommended that holding reservoir storage be incorporated in some new subdivisions for testing purposes. If criteria relating to the quantity and quality of runoff from new subdivisions were developed, the use of stormwater storage would be encouraged and the practical aspects of storage could be evaluated on a continuing basis.
2. It is recommended that the use of storage in industrial and commercial areas be investigated. Most industrial developments contain large impervious areas such as parking lots which produce high runoff rates and which may be suitable for temporary storage.
3. It is recommended that system studies for large drainage areas be undertaken. One useful study would be to determine the optimum location of holding reservoirs in the sewer network to minimize overflow pollution problems resulting from future urban growth.

1. INTRODUCTION

1.1 The Combined Sewer Overflow Problem

Combined sewers designed to accept both sanitary flows and stormwater flows have provided the means of wastewater collection in many urban areas of Canada for over 100 years. Conventionally, interceptor sewers with capacities 1.5 to 5 times greater than those required for dry weather flows are used to divert combined flows to a water pollution control plant for treatment before discharge to a water body. During storms, flows in excess of interceptor capacities are discharged directly to the receiving waters.

In recent years, there has been a growing concern with the pollution problems caused by combined sewer overflows. It has been estimated that by 1980 the average five-day BOD loadings on Lake Ontario from overflows in Metropolitan Toronto may reach 600 tons/month, almost one-half the anticipated total loading (1). As overflows will occur for only a few hours each month, the shock loadings generated by individual storms will be much greater than the average loading.

1.2 Combined versus Separate Sewers

Current practice favours separate sewer systems as a means of alleviating overflow pollution problems. The more critical sanitary wastes are collected, transported and treated separately so that discharges of raw sewage are eliminated. Generally, the storm sewer network is extended only to a suitable watercourse and stormwater flows are discharged without treatment.

For densely populated areas presently served by combined sewers, separation costs may be prohibitive. A study of a 17,000 acre area in the City of Toronto in 1960 indicated that the costs of sewer separation would be about \$17,000 per acre (2). For new developments in unsewered areas, there may be substantial savings in provision of a single conduit to serve as a combined sewer. The cost of a separate system for unsewered areas of Detroit has been estimated at twice the cost of a combined system (3).

The effectiveness of the separate system solution is largely dependent on the assumption that stormwater flows are relatively uncontaminated. But the concentration ranges of wastewater constituents from urban stormwater runoff have been found to be similar to those for combined sewer overflows (4). A study of a 27 acre residential area in Cincinnati compared the pollution loadings from stormwater runoff with estimated loadings from sanitary sewage on a yearly basis (5). Suspended solids (SS) contributions from stormwater runoff were found to be 40 per cent greater, and the BOD load was six per cent of that from raw sewage. A study of water quality in the upper Whippany River basin in New Jersey showed that the pollution loadings from unrecorded sources such as urban runoff were greater than those from secondary treatment plant effluents (6). Hence, inadequate water quality control plans may result if the pollution from stormwater runoff is not taken into account.

Research has been carried out to investigate the sources of pollution in urban runoff and the efficiency of "housekeeping" practices in reducing waste loads. Results from a study of street sweeping in Chicago indicated that, although machine sweepers remove most of the coarse material, they remove only a small proportion of fine material which is the main contributor to SS loadings (7).

Compatibility with the existing system is a further consideration when designing wastewater collection facilities for a new development. As urban areas expand, subdivisions are becoming increasingly remote from existing treatment facilities. Often, flows from a separate sanitary system in a subdivision will enter a trunk sewer which is a combined sewer and the potential benefits of a separate system will not be realized. Policies requiring treatment at large-scale downstream plants will also influence the design of collection systems for new developments (8).

A shortcoming of combined sewers is the accumulation of settled solids in the conduit during dry weather flow periods. Because of the wide range of flows in combined sewers, it is sometimes not practical to design the sewers to be self-cleansing for sanitary flows and storms must be relied upon to provide intermittent flushing.

Combined sewer systems preclude possibilities for beneficial use of stormwater. A recent study of an 1140 acre watershed in Columbia, Maryland, concluded that a system of small stormwater storage basins with treatment capability could provide approximately half the water demands of a typical urban residential community (9).

The best system of wastewater collection for any area will depend ultimately on the particular factors relevant to that area and on the existing system. In view of the multi-million dollar investment in existing combined systems, it would appear worthwhile to pursue investigation of some alternatives which incorporate combined sewers for servicing future urban development. But the feasibility of combined sewers is dependent upon the effectiveness of solutions for abatement of overflows during heavy storms. An extensive study of wastewater management needs in San Francisco recommended continuance of the practice of combined sewer construction, but with complementary control facilities having sufficient storage and treatment capacities to limit overflows to eight per year (10).

1.3 Use of Storage

Storage may be used to reduce peak flow rates and to control either combined sewer flows or separate storm sewer flows. Storage allows a greater proportion of the stormwater flows to be diverted for treatment without increasing interceptor or plant capacities.

The possible locations of storage in storm or combined sewer systems are shown schematically in Figure 1. The sources of inflow to sewers are stormwater runoff from rainfall, infiltration and, for combined sewers, sanitary flow.

Rainfall may be ponded directly on flat roofs; rooftop detention has been incorporated in office developments in Denver (11), for example. Alternatively, some form of on-site storage of runoff may be provided. Temporary surface ponding on residential lots has been adopted in El Paso (12). But this practice may not be acceptable where, for example, soils have low infiltration capacities. In some cases a constructed facility such as an underground tank may be feasible.

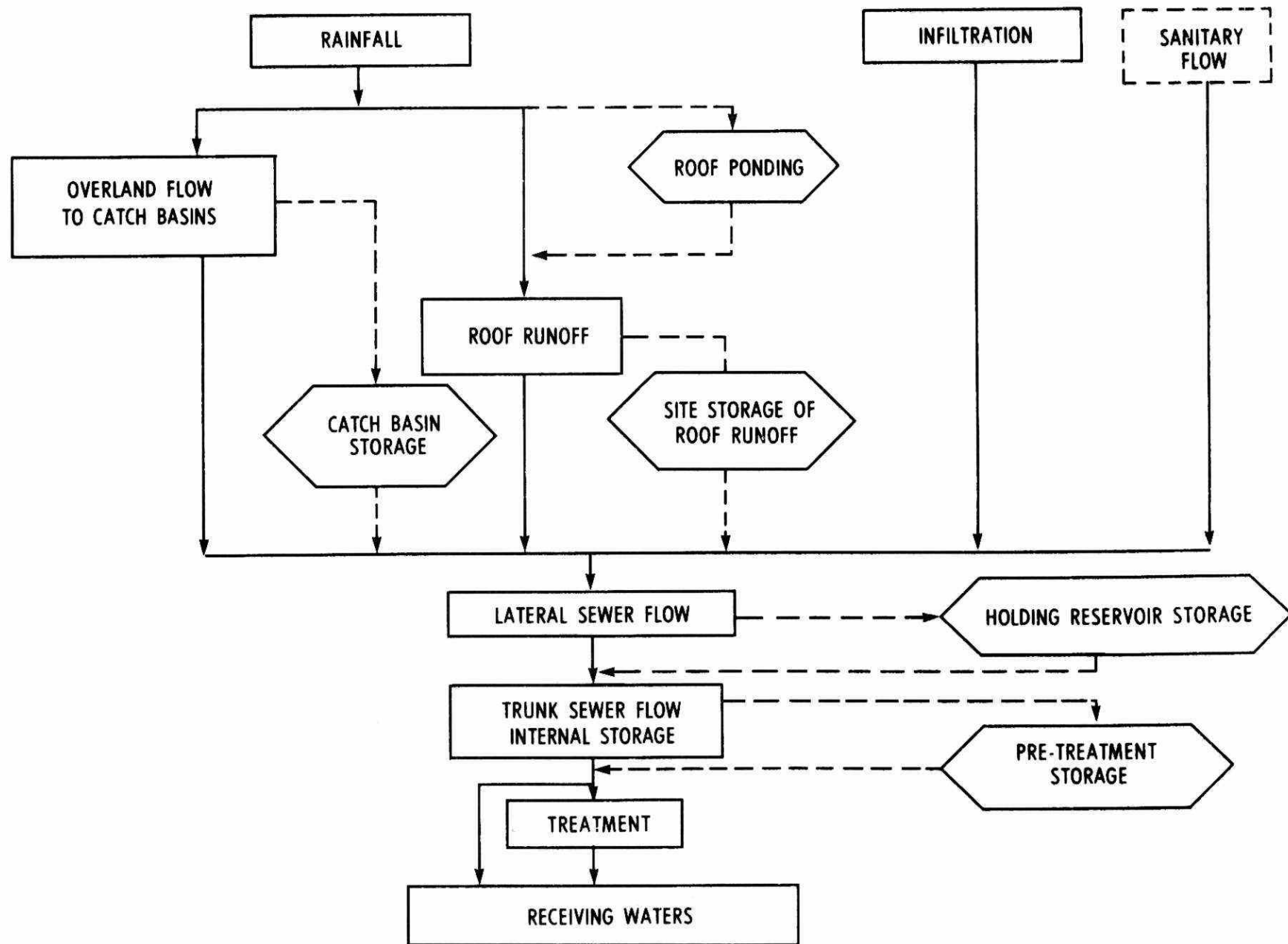


FIGURE 1. POSSIBLE LOCATIONS OF STORAGE IN STORM AND COMBINED SEWER SYSTEMS

A large part of the stormwater runoff enters sewers by way of catch basins into which street gutters discharge. The City of Toronto has undertaken a program of partial sewer separation to accommodate street runoff (2). A variation of this approach could incorporate storage at catch basins for temporary detention of street runoff before release to sewers.

Holding reservoirs may be incorporated in the sewer network to reduce peak sewer flows during storms. When detention times are sufficient for the reservoirs to operate as settling tanks, control of water quality may be a further benefit. Numerous cases of the use of storage tanks, deep tunnels, reservoirs, etc., have been presented in the literature and the storm stand-by tanks at Columbus, Ohio, are among the earliest examples (13).

The volumetric capacity of trunk sewers may be utilized for storage through monitoring and control of flow levels. A computerized control system has been implemented in Minneapolis-St. Paul to reduce the frequency and volumes of overflows from the existing combined sewer system (4).

Holding storage located before treatment facilities may improve treatment efficiencies as well as regulating flows. A full-scale study of a wastewater treatment plant at Newark, New York demonstrated that the addition of an equalization tank directly preceding the treatment units effected a substantial improvement in primary removal of suspended solids (14).

A computer-based stormwater management model has been developed by the U.S. Environmental Protection Agency (EPA) for testing alternative methods of alleviating existing combined sewer overflows (15, 16). Solutions incorporating sewer system storage options and/or treatment options are evaluated on a cost-effectiveness basis. A linear programming model has been formulated for use in conjunction with the EPA model to identify minimum cost solutions (17). A similar approach was used for selection of optimal combinations of stormwater holding reservoir locations, treatment types and water re-uses for a large urban watershed in Columbia, Maryland (9).

1.4 Storage in New Residential Subdivisions

Most research to date has been directed towards identification of existing combined sewer overflow problems and evaluation of methods for alleviating these problems. Methods of reducing stormwater flows from new developments seem to have received only limited attention.

The purpose of this research was to investigate the effectiveness of incorporating storage in sewer systems for new residential subdivisions. The storage methods to be examined were:

- (1) ponding of rainfall on flat roofs;
- (2) site storage of roof runoff;
- (3) storage of flows from street gutters at catch basins; and,
- (4) holding reservoirs as part of the sewer network.

A subdivision of approximately 100 acres which is currently under development in Southern Ontario served as a model for the study. The existing wastewater collection network for the subdivision consists of separate sanitary and storm sewers. The collection network was re-designed as a combined sewer system and the effects of storage were investigated for both the separate and combined systems.

A means of estimating stormwater runoff from the subdivision and of predicting the effects of storage on runoff rates was required. A survey of existing urban runoff models and the development of a hydrograph model suitable for the study are described in Chapter 2. In Chapter 3, a design storm is selected and the hydrograph model is used to determine storm sewer flows from the subdivision. The combined sewer design is summarized in Chapter 4 and combined sewer flows are determined. Routing functions for each of the storage methods are described in Chapter 5, and the effects of different storage options on sewer systems are investigated. In Chapter 6, costs of separate and combined sewer systems are evaluated for various storage options. The technical feasibility and economic attractiveness of the storage methods are discussed in Chapter 7 and some preliminary conclusions are made concerning the effectiveness of local storage.

2. ESTIMATION OF STORMWATER RUNOFF FOR RESIDENTIAL SUBDIVISIONS

2.1 Survey of Existing Urban Runoff Models

In order to estimate stormwater flows in sewers, it is necessary to determine the amount of runoff resulting from a given rainfall. Several urban runoff models have been developed for this purpose and a comparative analysis of 18 such models has recently been carried out (18). The objectives, advantages and limitations of each model were reviewed, and the performance of some of the models was tested using both real and hypothetical catchment data. A summary of four of the most widely accepted models follows.

2.1.1 Rational method

The rational method (19) is the simplest and most widely used method for estimating stormwater flows. The relationship between rainfall and runoff is expressed directly by the formula:

$$Q_{\text{peak}} = Ci_{\text{ave}}A$$

where: Q_{peak} is the peak rate of stormwater runoff (cfs);
 i_{ave} is the average intensity (in/hr) of rainfall of given frequency of occurrence having a duration equal to the time of concentration, which is the time required for runoff to flow from the remotest part of the drainage basin to the point under design;
 A is the area of the drainage basin (acres); and,
 C is the runoff coefficient which is the ratio of peak runoff rate to average rainfall rate.

Rainfall intensities are obtained from average intensity - duration - frequency relationships which are of a form such as:

$$i_{\text{ave}} = \frac{a}{t_d^b + c}$$

where: t_d is the rainfall duration (min) equal to the time of concentration; and,

a, b, c are constants.

For storm sewers in residential areas, the average frequency of rainfall occurrence used for design commonly varies from two to ten years. In effect the rational method assumes an idealized uniform intensity storm over the duration t_d .

The runoff coefficient C attempts to account for many variables, including infiltration, ground slope, depression storage, etc. The effects of antecedent precipitation cannot be easily evaluated; reasonable estimation of C values requires experience and good judgement.

The rational method has been found to give adequate estimates of peak runoff rates for urban areas of less than five sq miles. But because the variation of runoff with time is not identified, it cannot be used for design of systems incorporating storage.

2.1.2 Road Research Laboratory (RRL) method

As a first step to generating a sewer flow hydrograph, the RRL method (20) may be of some value. The drainage area is subdivided into a number of zones within which the time of concentration is constant. A rainfall intensity is determined for each time increment and ordinates of a simulated sewer inflow hydrograph are computed using a direct rainfall-runoff formula. The inflow hydrograph is added to the hydrograph from up-stream, and the resultant hydrograph is lagged by the time of travel through the sewer at full-flow velocity to the next stormwater inlet, and so on.

A possible limitation of this method is that there is assumed to be no runoff from pervious areas. For low density residential areas with relatively small impervious areas and high intensity storm, the RRL method may give unrealistically low runoff estimates.

2.1.3 Chicago hydrograph method

The Chicago hydrograph method (21) isolates and evaluates all the influences which transform a hyetograph (rainfall intensity - time pattern) into a sewer flow hydrograph. A design storm pattern is selected

and the drainage basin is schematized into pervious and impervious areas. For roofs which are directly connected to sewers, the roof runoff hydrograph immediately becomes a sewer inflow hydrograph. For rain which falls on the ground, the steps in the method are:

- Step 1: estimation of abstractions from rainfall
(infiltration and surface depression storage)
- Step 2: calculation of overland flow to street gutters
- Step 3: flow routing through street gutters to catch basins

For calculating overland flow, an experimentally derived equation relating flow depth, surface slope and discharge is used. Gutter inflow is assumed to be uniformly distributed along the length of the gutter.

Inflow hydrographs from roofs and catch basins are combined to form a sewer flow hydrograph which is routed through the sewer using a simple time offset method: the hydrograph is moved along the sewer at the sewer full-flow velocity.

2.1.4 U.S. Environmental Protection Agency Storm Water Management Model

The EPA model contains a runoff sub-model (22) which simulates the fundamental processes of runoff phenomena through a computer program. These processes are similar to the influences examined in the Chicago hydrograph method but they are treated somewhat differently.

A separate roof runoff hydrograph is not identified. Overland flow is modelled as a quasi-steady state phenomenon using Manning's equation for uniform flow. Gutter inflow is assumed to occur only at the upstream end of a gutter, and sewer flow is modelled as unsteady, non-uniform flow.

2.2 A Hydrograph Model for Residential Subdivisions

From the survey of existing urban runoff models, it was concluded that none of the models was suitable for application to the present study, particularly with respect to evaluation of the proposed storage alternatives. Hence it was decided to develop a model which directly incorporated functions to represent the different storage methods. Certain features of both the Chicago hydrograph method and the EPA model were included.

2.2.1 Model development

A flow chart of the hydrograph model developed for investigation of storage effects in sewer systems for residential subdivisions is shown in Figure 2. A listing of the computer program in which the model is incorporated is provided in Appendix II.

Input requirements for the model are a design storm in the form of a hyetograph and a schematization of the subdivision. The drainage area is represented as a set of hydraulic elements: rectangular subcatchments based on pervious and impervious areas; gutter and sewer networks; and, storage elements.

The runoff from directly connected roofs and the overland flow into street gutters are computed independently. For urban catchments, infiltration and depression storage are the dominant abstractions from rainfall. There is assumed to be no infiltration in impervious areas, and for pervious areas, Horton's exponential function (21) is used:

$$f = f_c + (f_o - f_c) e^{-kt}$$

where: f is infiltration capacity (in/hr) at time t ;
 t is time from beginning of infiltration capacity curve (min);
 f_c is minimum infiltration rate as time t approaches infinity;
 f_o is maximum infiltration rate at time zero;
 k is decay rate of infiltration capacity (min^{-1}).

Overland flow from a subcatchment commences when the depth of ponding exceeds the average depression depth of the subcatchment. For computing overland flow and gutter flow into catch basins, a stepwise procedure incorporating Manning's equation for uniform flow is used. The procedure is similar to that used in the EPA model and a full description is provided in reference 22. The steps in the computation of overland flow from a subcatchment are as follows:

1. The depth of water after rainfall is calculated:

$$D_1 = D_t + R_t \Delta t$$

where: D_1 is water depth after rainfall occurs

D_t is water depth at time t

R_t is rainfall intensity in time interval Δt

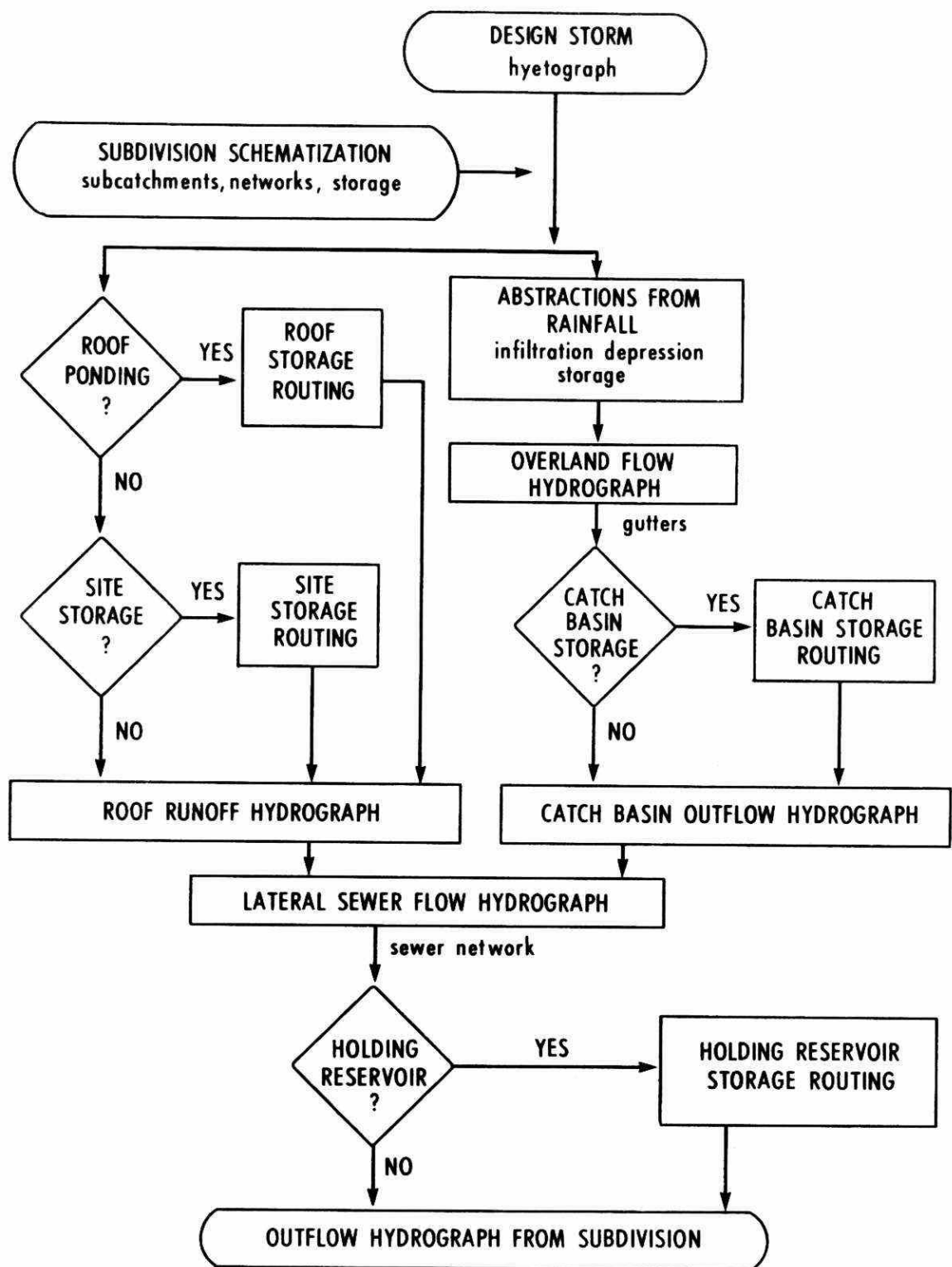


FIGURE 2. FLOW CHART FOR SUBDIVISION HYDROGRAPH MODEL

2. Infiltration is subtracted from the water depth (for pervious areas):

$$D_2 = D_1 - f \Delta t$$

where D_2 is water depth after allowing for infiltration.

3. If the depth of water is larger than the specified depression depth, the flow rate is computed from Manning's equation:

$$Q_w = \frac{1.486}{n} (D_2 - D_d)^{5/3} S^{1/2} W$$

where: Q_w = outflow rate

n = Manning's coefficient

D_d = depression depth

S = ground slope

W = width of subcatchment

4. The depth of water after outflow is calculated:

$$D_t + \Delta t = D_2 - \frac{Q_w}{A} \Delta t$$

where: A = surface area of subcatchment.

5. Steps 1 to 4 are repeated for successive time intervals.

Gutter flows are calculated in a similar manner and catch basin hydrographs are computed.

Roof runoff hydrographs, which are computed directly from the hyetograph, and catch basin outflow hydrographs are combined to form lateral sewer flow hydrographs which are routed through the sewer network using the time offset method. The principal output from the model is a sewer flow hydrograph at the outfall from the subdivision.

Routing functions for each of the storage alternatives are built into the model at the appropriate locations. The precise form of these functions is explained in Chapter 5.

2.2.2 Model calibration

When applying the model to any drainage catchment, values of catchment parameters such as infiltration capacities and average depres-

sion depths must be specified so that realistic estimates of stormwater flows are obtained. Where synchronized rainfall and runoff records for the catchment exist, the calibration process consists of adjusting parameter values to achieve good fit between hydrographs calculated from the model and measured hydrographs.

For a newly sewered catchment, parameter values might be obtained from catchments with similar characteristics for which values are known. Alternatively, when no information is available, a set of default values has been suggested for use in conjunction with the EPA model and these values were incorporated in the original version of that model (16).

2.2.3 Model testing

The model was tested by comparing the outflow hydrographs predicted by the model with the measured hydrographs for the Gray Haven catchment which is located near Baltimore, Maryland. The land use is residential and the catchment has an area of 23.3 acres with average imperviousness of 52 per cent. Rainfall intensities and runoff at a stormwater outfall have been recorded for a number of storms (23).

The model was used to calculate runoff from four storms, two of high rainfall intensity and two of lower intensity. Details of the storms are given in Table 1. The values of three characteristic parameters of the outflow hydrograph were calculated:

1. peak runoff rate (cfs) - Q_{peak}
2. time to runoff peak (min) - T_{peak}
3. total runoff volume (cu ft) - Vol

The values predicted by the model are compared with the measured values in Table 2 and the complete hydrographs for the storm of June 10, 1963 are shown in Figure 3. The subdivision hydrograph model gave good results for the high intensity storms and fair results for the less intense storms.

2.2.4 Limitations of the model

The hydrograph model was developed with a view solely to its use in the subdivision study. Hence, its general applicability may be

Table 1
Details of Test Storms for Gray Haven Catchment

Storm No.	Date of Storm	Av. Intensity During Peak 10 min. Period (in/hr)	Total Rainfall (in)
1	5/6/1963	4.7	1.77
2	10/6/1963	4.7	2.20
3	14/6/1963	2.8	1.40
4	20/6/1963	2.8	1.46

Table 2
Comparison of Measured and Predicted Values for
Runoff Hydrographs from Gray Haven Catchment

Storm No.	Measured Values	Predicted Values	
		Subdivision Model	Hydrograph
	Q_{peak} (cfs)	Q_{peak} (cfs)	Error (%)
1	80.7	80.3	-0.5
2	79.0	80.8	2.3
3	30.8	32.7	6.1
4	29.6	35.7	20.6
	T_{peak} (min)	T_{peak} (min)	Error (%)
1	41	39	-4.9
2	41	39	-4.9
3	22	21	-4.5
4	22	21	-4.5
	Vol. (cu ft)	Vol. (cu ft)	Error (%)
1	127,400	129,900	2.0
2	123,900	129,900	4.8
3	51,200	60,100	17.4
4	49,000	61,200	25.0

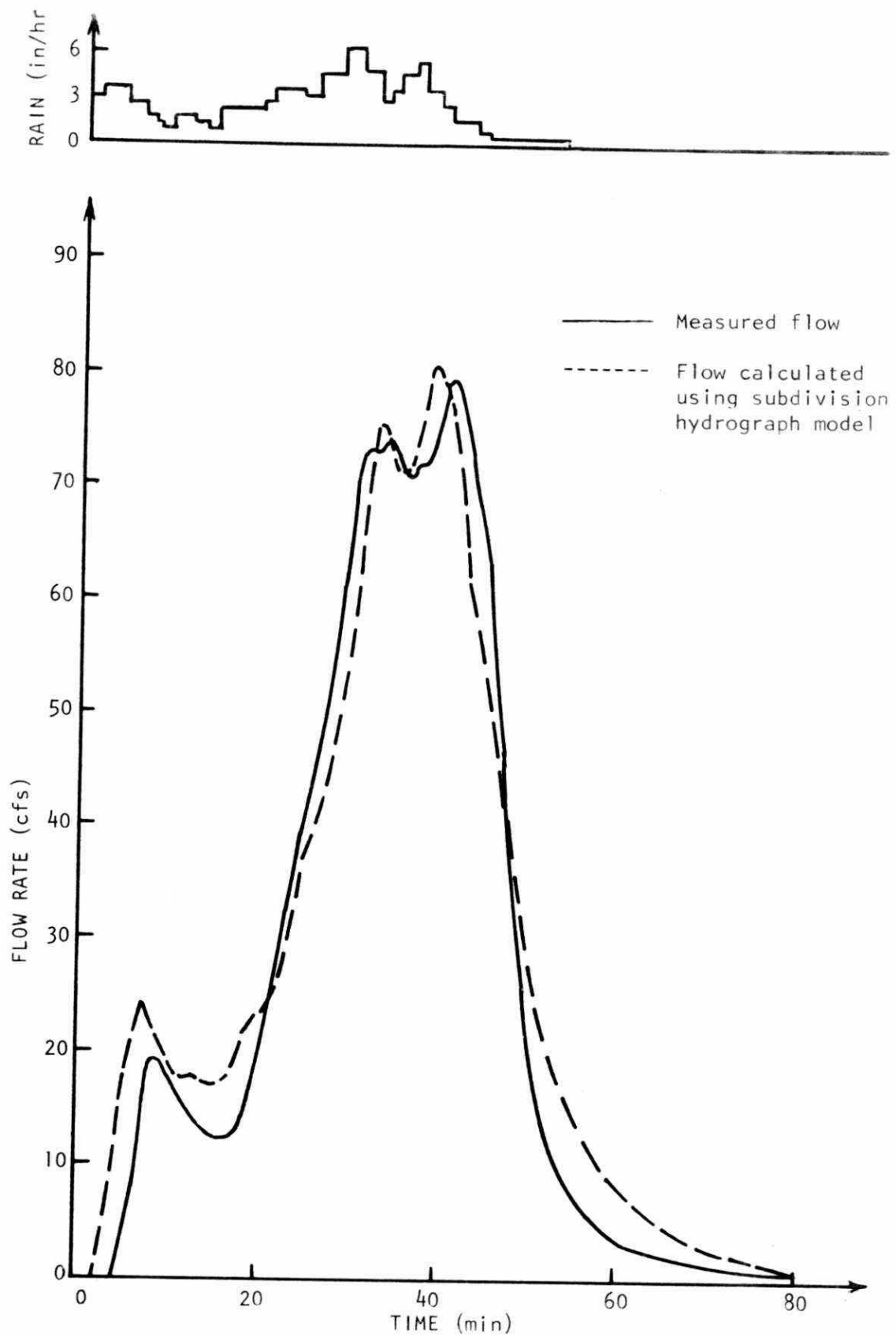


FIGURE 3. COMPARISON OF MEASURED AND CALCULATED RUNOFF HYDROGRAPHS FOR GRAY HAVEN CATCHMENT - STORM OF JUNE 10, 1963.

limited. The most serious limitation of the present form of the model is that in the schematization of the drainage area, all hydraulic elements must be identified. Roof areas must be separated from other impervious areas such as streets; the pervious and impervious subcatchments must be identified for each gutter; and, the locations of each catch basin and each sewer must be specified. As a result, the data requirements and computer core usage are very large, even for a small test area. In order to evaluate the local storage methods proposed for this study, such a detailed schematization was necessary.

For more general studies involving large drainage areas, some aggregation of subcatchments is desirable. Measured runoff hydrographs have been reproduced successfully using the EPA model with coarse subcatchment data. The performance of the subdivision hydrograph model in this respect has not been tested.

With the present version of the model, the time step (Δt) in the computational procedure is required to be quite small for a stable solution to be obtained. A time step of 0.5 minutes was used for this study. A numerical technique such as the Newton-Raphson method could be incorporated in the model to provide a stable solution for longer time steps, enabling computing costs to be reduced. Time steps of five minutes are used in typical applications of the EPA model, which employs the Newton-Raphson method.

3. ESTIMATION OF STORM SEWER FLOWS FOR STUDY AREA

3.1 Schematization of Subdivision

A plan of the subdivision which was subjected to detailed study is shown in Figure 4, along with the layout of the existent separate sewer network. The primary land use is residential with 383 single family lots. As well, there are two small industrial areas, two parks and a school. The areas devoted to each land use are:

	<u>Acres</u>
residential area	83.2
industrial area A	5.2
industrial area B	4.2
school	1.1
park A	1.8
<u>park B</u>	<u>6.1</u>
Total	101.6

The sewer network consists of 56 individual links between man-holes. The sewers in the line from the uppermost part of the catchment to the outfall have been numbered from 1 to 18. For clarity in presentation, the only other numbered sewer is sewer no. 56 which connects the top part of the catchment to the outfall.

The original storm sewer design was carried out using the rational method with the following runoff coefficients:

residential areas	0.45
industrial areas	0.70
school	0.60
parks	0.20

Details of the existing storm sewers are provided in Table 3. Except for storm sewer no. 1, the full-flow velocities are well in excess of the desirable minimum of 3 ft/sec (19). The average depth was calculated for cost estimation purposes and represents the distance from one foot below the finished road surface level to the sewer invert level.

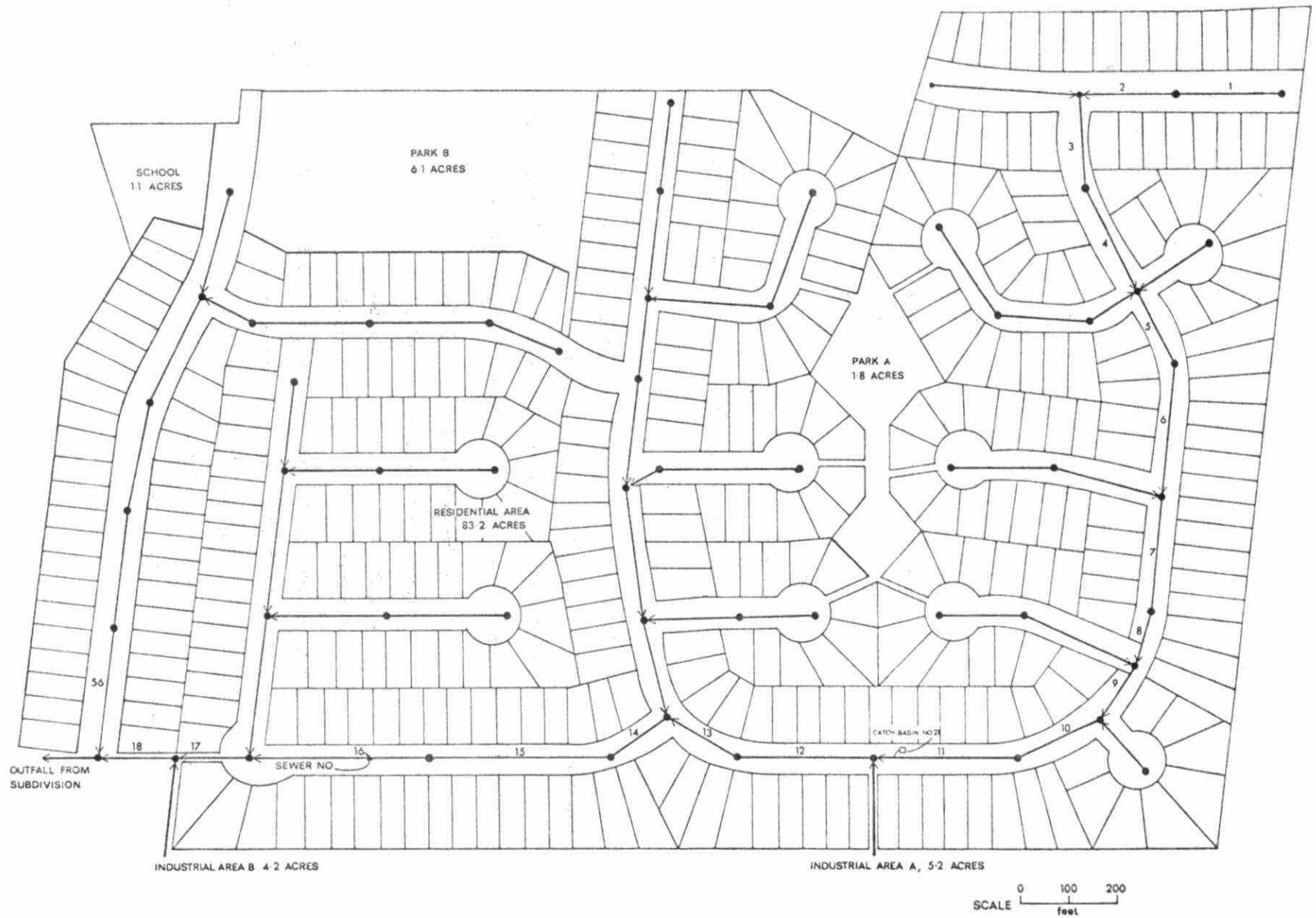


FIGURE 4. PLAN OF SUBDIVISION.

Table 3

Details of Existing Storm Sewers

Storm Sewer No.	Diameter (in)	Full-flow Velocity (ft/sec)	Length (ft)	Flow Time (min)	Average Depth (ft)
1	15	2.9	230	1.3	13.1
2	18	3.2	223	1.2	12.5
3	21	3.6	214	1.0	11.7
4	21	3.6	236	1.0	13.0
5	27	4.3	162	0.6	15.9
6	30	4.6	272	1.0	17.4
7	33	4.9	245	0.8	17.5
8	33	4.9	110	0.4	16.8
9	36	5.2	142	0.5	16.6
10	36	5.2	196	0.6	15.9
11	36	5.2	300	1.0	14.8
12	42	5.2	295	0.9	14.8
13	42	5.2	158	0.5	16.4
14	48	5.7	145	0.4	18.5
15	48	5.7	375	1.1	20.7
16	48	5.7	375	1.1	21.0
17	54	6.8	145	0.3	18.6
18	54	6.8	160	0.4	18.6
Total			3983	14.1	

With the subdivision hydrograph model, a detailed schematization of the subdivision drainage area is required, and Figure 5 shows the schematization for storm sewer no. 1. The average roof area per lot is 1,700 sq ft and there are 10 roofs connected directly to the sewer. The other impervious areas are the street, which is 19 feet from crown to gutter, with two per cent cross-slope and lot driveways which are 10 feet wide and 49 feet long. Facilities such as walkways which drain on to lawns are considered as pervious areas. With this schematization, the average imperviousness of the 83.2 acre residential area is 34 per cent - 18 per cent roofs, and 16 per cent streets and driveways.

From Figure 5, it can be seen that because of the direction of slope of the street gutters, only 115 feet of the gutter contributes flow to storm sewer no. 1 via catch basins no. 1 and 2. The remaining flow drains to catch basins no. 3 and 4 which connect to storm sewer no. 2. The total area contributing to storm sewer no. 1 is 1.14 acres with 56 per cent imperviousness, which is relatively high because of the large proportion of roof area.

The irregularly shaped pervious areas which drain from rear to front are represented by equivalent rectangular subcatchments with the following dimensions:

$$\text{width} = \text{gutter length} - \text{total driveway width}$$

$$\text{length} = \text{pervious area}/\text{width}$$

The subcatchment width is the dimension perpendicular to the direction of overland flow and the subcatchment length is the dimension in the direction of flow. For the pervious area of 10,700 sq ft contributing to catch basin no. 2, the subcatchment dimensions are:

$$\text{width} = 115 - 20 = 95 \text{ feet}$$

$$\text{length} = 10,700/95 = 112 \text{ feet}$$

Since the subcatchment length is somewhat less than the distance from the rear of a lot to the street gutter (144 feet), the runoff from back lawns may be overestimated.

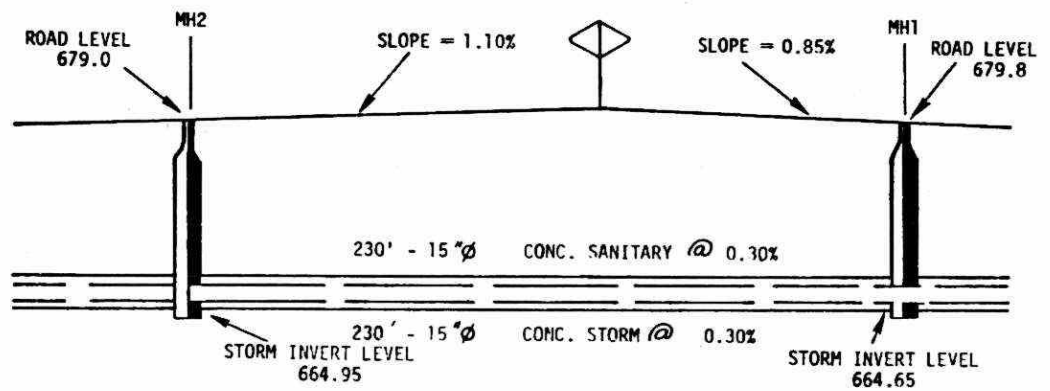
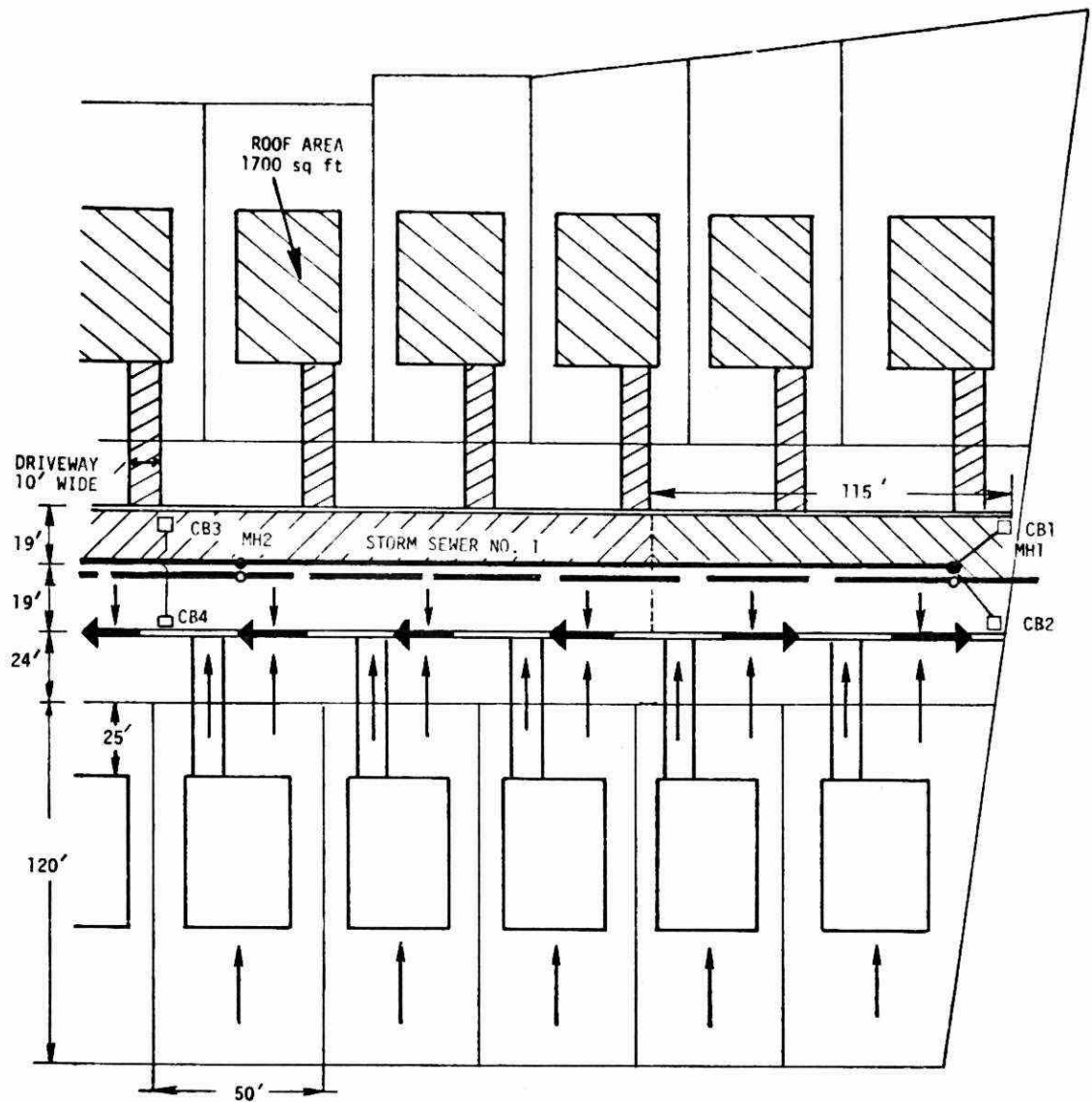


FIGURE 5. DRAINAGE PLAN AND SEWER PROFILE FOR STORM SEWER NO. 1

3.2 Selection of Design Storm

The choice of return period of the rainfall event used for storm or combined sewer design is based on a trade-off between the costs of providing sewers and the costs of flooding resulting from stormwater flows greater than sewer capacities. With rational method design, an average intensity - duration curve for the selected return period is used, whereas hydrograph methods require the use of a particular rainfall intensity-time pattern. It is generally possible to derive average intensity-duration relationships from the precipitation record for any area, but a definite return period cannot be assigned to individual recorded storms. In practice, a synthetic design storm which satisfies the average intensity - duration relationship for the selected return period can be derived from a statistical analysis of recorded storms.

The original design of the storm sewers, using the rational method, was based on a two-year frequency storm with the following average intensity - duration formula:

$$i_{ave} = \frac{44}{t_d^{0.8} + 8}$$

This formula is shown graphically in Figure 6.

A study of storms in the Chicago area indicated that there was a consistent variation of rainfall intensity with time, with peak intensity located at approximately three-eighths of the total storm duration (21). A design storm of 180 minutes duration with a peak at 67.5 minutes was adopted for sewer design in Chicago. Since no rainfall intensity-time data were available for the study area, a synthetic design storm of similar form to that for Chicago was assumed. The rainfall intensity values were adjusted to provide conformity to the average intensity-duration formula used in the original storm sewer design for the subdivision. The equations used for the design storm pattern were:

before the peak:
$$i = \frac{44 [0.2(8/3 t_b)^{0.8} + 8]}{[(8/3 t_b)^{0.8} + 8]^2}$$

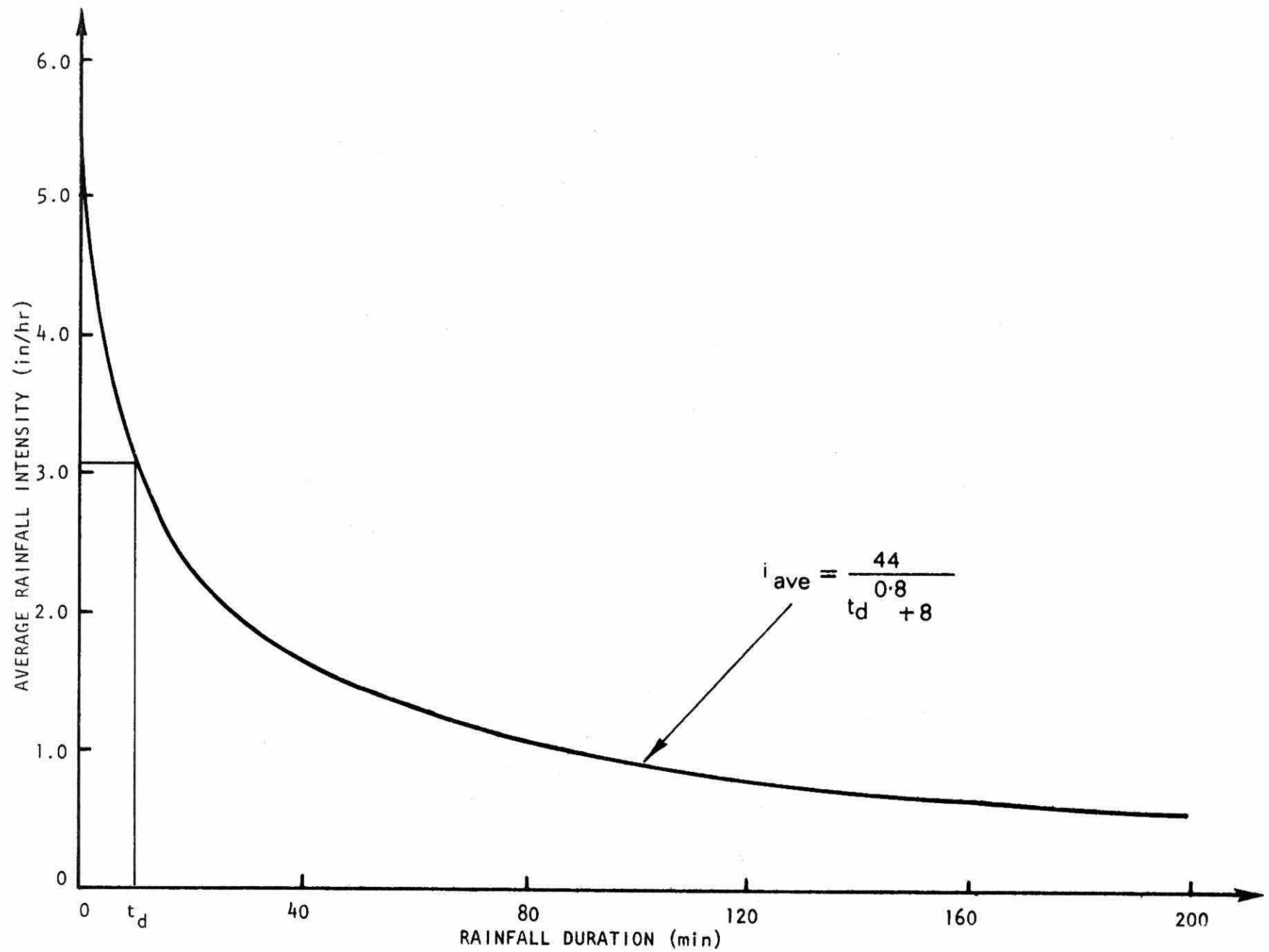


FIGURE 6. AVERAGE INTENSITY - DURATION CURVE FOR DESIGN STORM

after the peak:
$$i = \frac{44 [0.2(8/5 t_a)^{0.8} + 8]}{[(8/5 t_a)^{0.8} + 8]^2}$$

where: i is rainfall intensity (in/hr);
 t_a is time before peak (min); and,
 t_b is time after peak (min).

The design storm pattern is shown in Figure 7, together with the average intensity over the critical duration (t_d). The critical duration is located in the period of most intense rainfall and the value of t_d equals the time of concentration for rational method design.

3.3 Storm Sewer Flows for Uniform Intensity Storms

Since no data were available which would enable calibration of the subdivision hydrograph model to the study area, it was decided to adjust catchment parameter values to give correspondence between peak storm sewer flows calculated using the model and those calculated using the rational method.

Idealized uniform intensity storms were assumed for input hydrographs. Intensity values for a given storm duration were obtained from the average intensity - duration formula.

Whereas antecedent conditions are not considered explicitly with the rational method, the rising limb of the design storm can be utilized to establish antecedent conditions for the critical period. For pervious areas, the infiltration capacity does not drop below the rainfall intensity until the accumulated mass of rainfall exceeds the accumulated mass of infiltration capacity. This condition is represented by the equality of the hatched areas in Figure 8 for an infiltration capacity curve of the form:

$$f = 0.53 + (2.00 - 0.53)e^{-0.07t}$$

Reduction of infiltration capacity occurs after 43 minutes and ponding of water on pervious areas begins after 58 minutes when the rainfall intensity (i) exceeds the infiltration capacity (f). The cross-hatched area is a measure of the depth of ponding at the start of the critical period. For impervious areas, depression storage is filled at the start of the critical period.

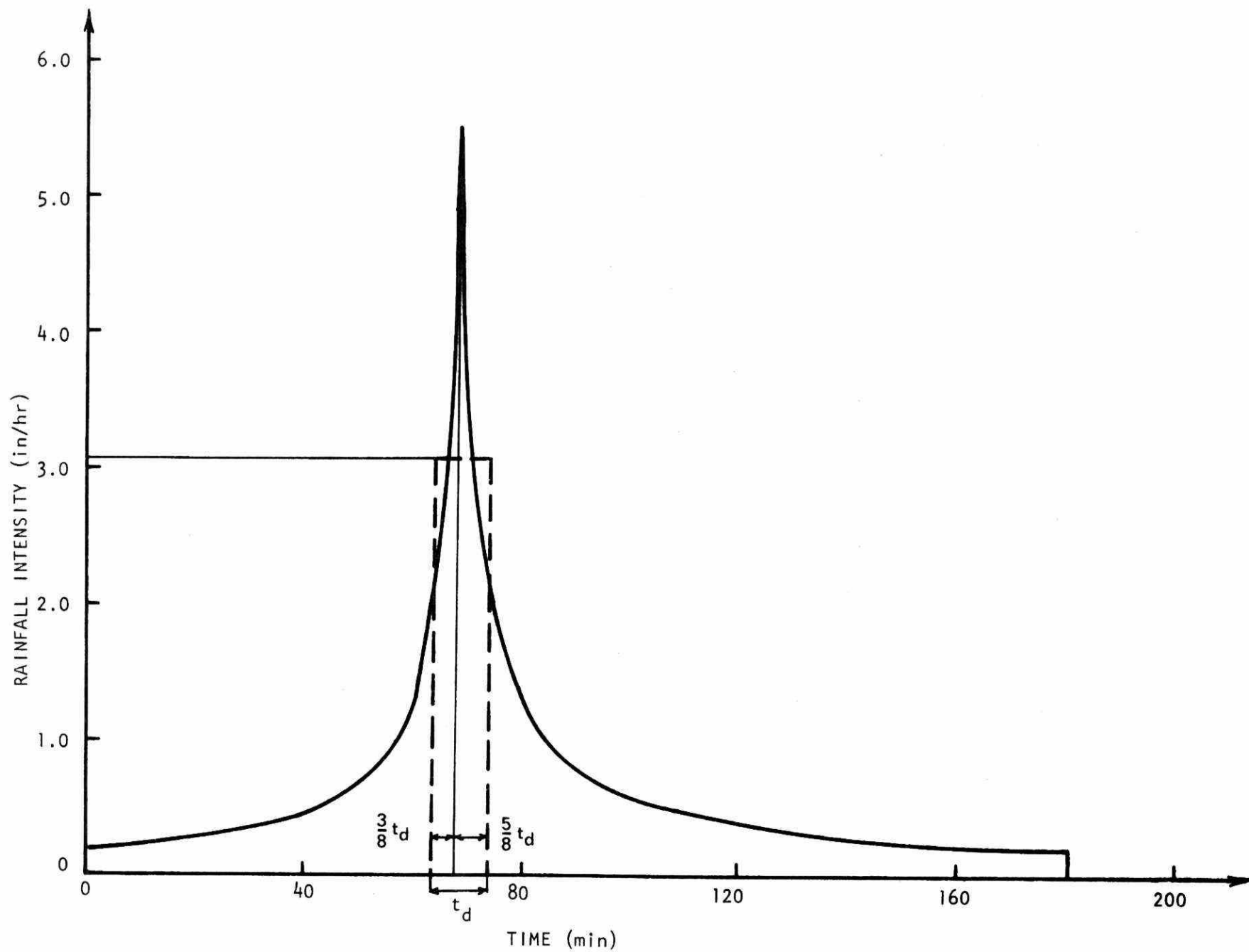


FIGURE 7. RAINFALL INTENSITY - TIME PATTERN FOR DESIGN STORM

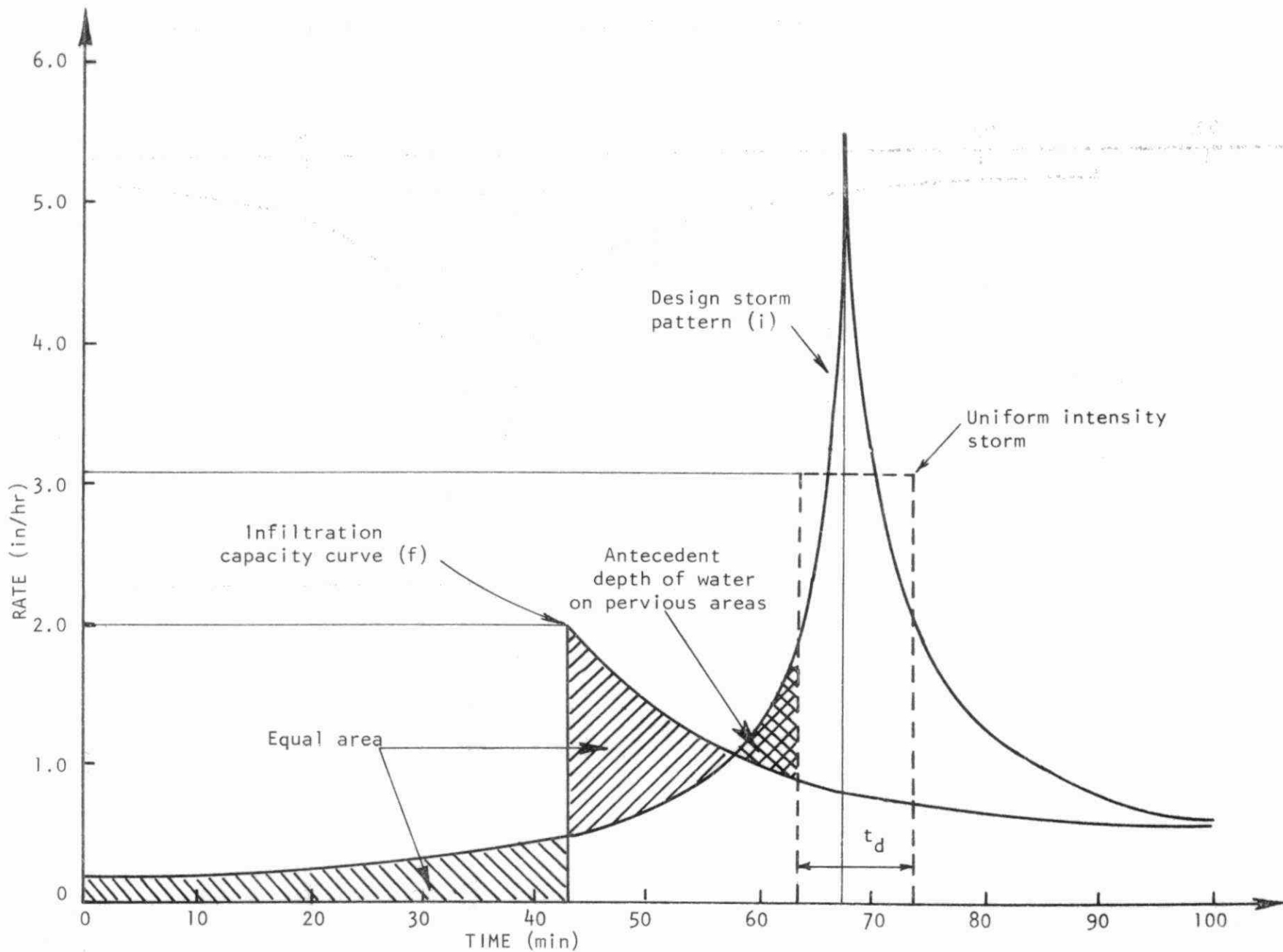


FIGURE 8. DETERMINATION OF ANTECEDENT CONDITIONS FOR UNIFORM INTENSITY STORM

Uniform intensity storms of different durations were tested at time intervals of 0.5 minutes for each of the storm sewers. A set of values of catchment parameters, found to give reasonable correspondence between peak flows from the hydrograph model and those from the rational method, is listed in Table 4. The recommended default values for the EPA model are shown in brackets.

For storm sewer no. 1, based on the hydrograph model, a 3.5 minute storm with uniform intensity of 4.10 in/hr gives the maximum peak flow of 2.17 cfs; the corresponding sewer flow hydrograph is shown in Figure 9. Stormwater flow at the downstream end of the sewer commences with the first contribution from directly connected roofs. A time lag of 0.5 minutes has been assumed for roof runoff to reach the sewer. The flow from roofs increases linearly as the contributing area increases, and all roofs are contributing after 1.8 minutes, which is the sum of the lag time and travel time from the upstream end of the sewer. The roof runoff remains constant until the end of the storm, after which it decreases linearly. The runoff from catch basins does not commence until overland flow and gutter flow are established. The runoff rate increases as water accumulates on ground surfaces and in gutters, and at the end of the storm, the runoff recedes as flow depths decrease. The original rational method design for storm sewer no. 1 used a time of concentration of 7.0 minutes (for which $i_{ave} = 3.45$ in/hr), a drainage area of 1.61 acres, and a C value of 0.45 to derive a peak flow of 2.50 cfs.

From the hydrograph model, the variation of peak flow estimate with rainfall duration is illustrated in Figure 10 for storm sewer no. 11, which has a contributing area of 29.7 acres with average imperviousness of 34 per cent. For storm durations longer than about nine minutes, the peak flows from roofs decrease while the flows from catch basins increase because of greater overland flow from pervious areas. The peak sewer flow, i.e. the sum of flows from catch basins and roofs, has a maximum value of 32.80 cfs for a storm of 11.5 minutes duration. The estimated time of concentration for the rational method design of storm sewer no. 11 was 15.4 minutes and Figure 10 shows that a storm of this duration would produce a slightly lower peak flow.

Table 4

Catchment Parameter Values for Subdivision

Parameter Description	Subdivision Hydrograph Model Values	EPA Model Default Values
Ground slope	4%	(3%)
Impervious areas:		
Manning roughness	.013	(.013)
Average depression depth	1/16 in	(1/16 in)
Pervious areas:		
Manning roughness	.200	(.250)
Average depression depth	3/16 in	(3/16 in)
Infiltration:		
Maximum rate	2.00 in/hr	(3.00 in/hr)
Minimum rate	0.53 in/hr	(0.52 in/hr)
Decay rate	0.070 min ⁻¹	(0.069 min ⁻¹)
Gutters:		
Manning roughness	.016	(-)
Side slope	5%	(-)

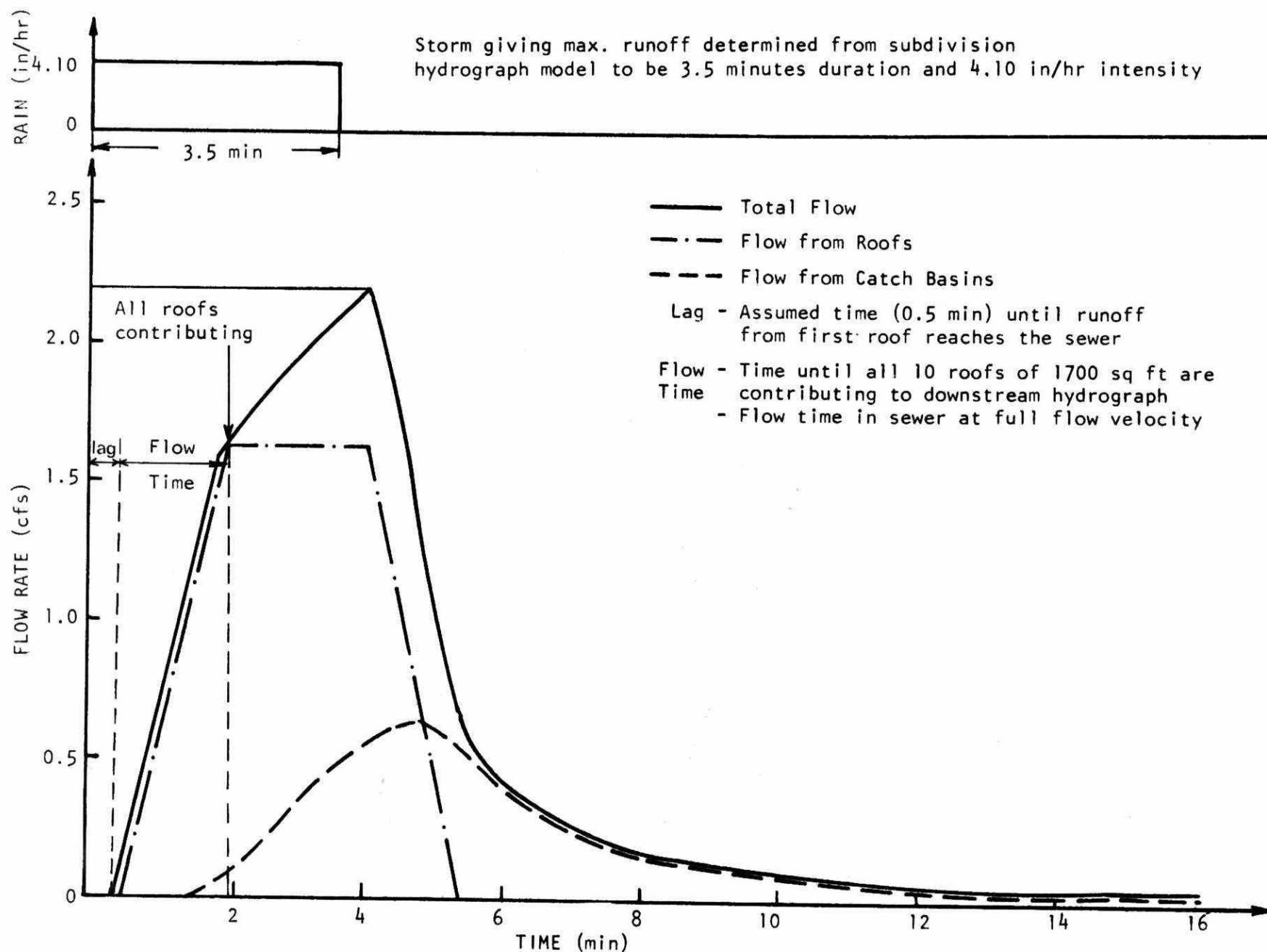


FIGURE 9. HYDROGRAPH AT DOWNSTREAM END OF STORM SEWER NO. 1 FOR 3.5 MINUTE STORM OF UNIFORM INTENSITY

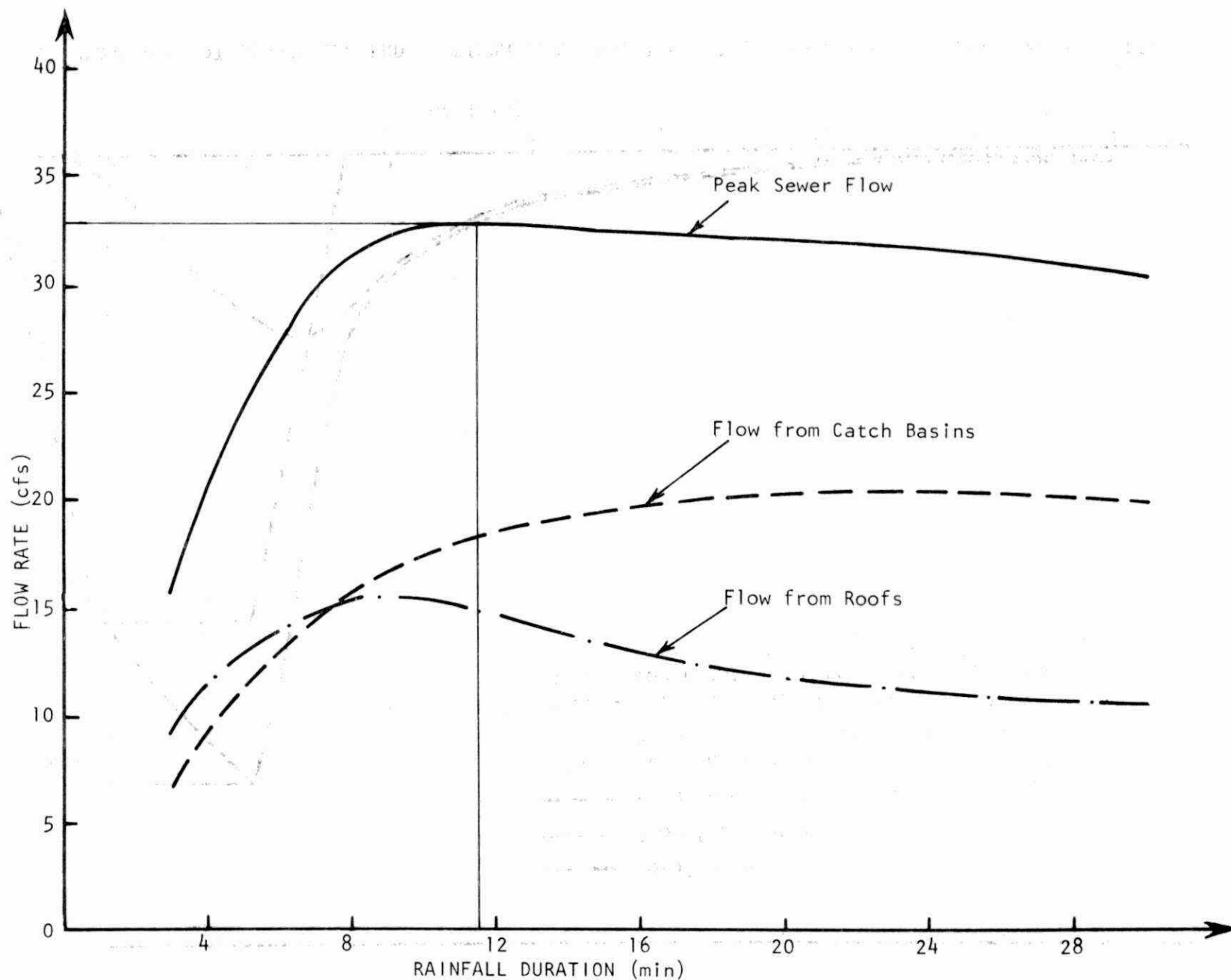


FIGURE 10. PEAK SEWER FLOW vs RAINFALL DURATION FOR STORM SEWER NO. 11

A sensitivity analysis was carried out to determine the effects on peak flow in storm sewer no. 11 of changing values of catchment parameters. The results are summarized in Table 5. The peak flow is most affected by changes in parameters influencing runoff from pervious areas: Manning roughness, depression depth and infiltration capacity. The peak flow is quite insensitive to changes in roughness and depression depth for impervious areas because runoff is well established at the start of the critical period. The amount of directly connected roof area has a significant influence on peak flow.

For non-residential areas where detailed information concerning land use and drainage was not available, a simplified stormwater runoff hydrograph of the form shown in Figure 11 was assumed. The apex of the hydrograph is set by the peak flow estimate from the rational method and both the rising and recession limb are linear, with length equal to the storm duration (t_d). Since the area under the hydrograph is a measure of total runoff, this hydrograph form assumes that the runoff coefficient C also represents the ratio of volume of runoff to volume of rainfall. Such idealized hydrographs have previously been used to investigate the effects of storage when only peak flow estimates were available (24).

3.4 Storm Sewer Flows for Design Storm

Once a set of catchment parameter values was established, the concept of an idealized uniform intensity storm was discarded and the design storm pattern (Figure 7) was adopted as the input hyetograph. To reduce the problem to a manageable size, the rainfall - runoff process was not simulated over the full 180 minute rainfall duration. Instead, the time origin was fixed at 43 minutes (when infiltration capacity begins to decrease), and an 80 minute rainfall was simulated. With this time origin, the rainfall peak occurred after 24.5 minutes. The intense rainfall was contained in the 80 minute period and since the time of flow through the sewer network was only 14.1 minutes, this period was adequate for the purposes of the study.

Because of the varying rainfall intensity - time pattern, the shape of the triangular runoff hydrograph for non-residential areas was changed. The peak flow was assumed to occur at a time after the peak

Table 5
Sensitivity Analysis for Peak Flow
in Storm Sewer No. 11

Parameter Description		Peak Flow (cfs)	% Change in Peak Flow
BASE RUN		32.80	--
Ground slope	= 2%	31.84	-2.9
Ground slope	= 6%	33.68	+2.7
Impervious areas:			
Manning roughness	= .011	32.83	+0.1
Manning roughness	= .015	32.77	-0.1
Ave. depression depth	= 0	32.80	0.0
Ave. depression depth	= 1/8 in	32.80	0.0
Pervious areas:			
Manning roughness	= .150	34.13	+4.0
Manning roughness	= .300	31.78	-3.1
Ave. depression depth	= 1/8 in	36.02	+9.8
Ave. depression depth	= 1/4 in	31.00	-5.5
Infiltration:			
Maximum rate	= 1.50 in/hr	34.76	+6.0
Maximum rate	= 3.00 in/hr	30.26	-7.7
Gutters:			
Manning roughness	= .020	32.19	-1.9
Manning roughness	= .024	31.73	-3.3
Side slope	= 3%	32.38	-1.3
Side slope	= 7%	33.04	+0.7
Average roof area			
	= 1,500 sq ft	31.02	-5.4
	1,900 sq ft	34.66	+6.0

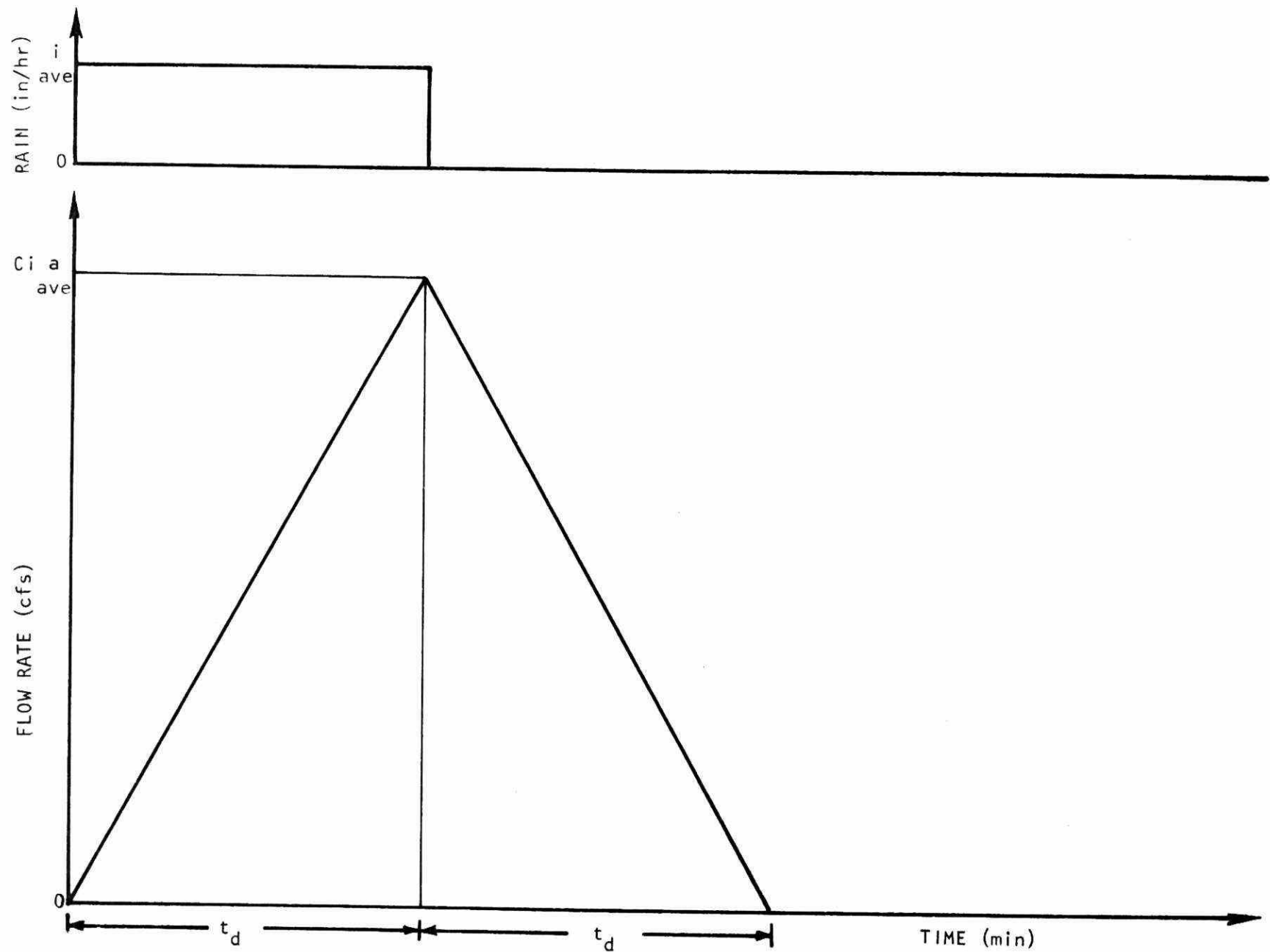


FIGURE 11. IDEALIZED TRIANGULAR RUNOFF HYDROGRAPH FOR UNIFORM INTENSITY STORM
(NON-RESIDENTIAL AREAS)

rainfall equal to the time of flow from the non-residential area to the sewer network, and the recession limb was assumed to be 1.67 times as long as the rising limb. With this assumption, the peak runoff was located at three-eighths of the total runoff duration. Since the peak rainfall is located at three-eighths of the total rainfall duration, the assumption should be reasonably accurate. Figure 12 illustrates the idealized runoff hydrograph for industrial area A, for which the peak flow estimate from the rational method is 11.4 cfs and estimated flow time to the sewer network is 2.5 minutes.

The area under the hydrograph gives a measure of the total volume of runoff.

Total volume of runoff from industrial area A

$$= 1/2 \times 11.4 \text{ cfs} \times 72 \text{ mins} \times \frac{60 \text{ sec}}{1 \text{ min}} = 24,600 \text{ cu ft}$$

The volume of rainfall on industrial area A

$$= 1.84 \text{ in} \times \frac{1 \text{ ft}}{12 \text{ in}} \times 5.2 \text{ acres} \times \frac{43,560 \text{ sq ft}}{1 \text{ acre}} = 35,500 \text{ cu ft}$$

$$\frac{\text{Volume of runoff}}{\text{Volume of rainfall}} = \frac{24,600}{35,500} = 0.69$$

The total volume of runoff with the assumed hydrograph agrees closely with that which would be obtained using a runoff coefficient of 0.70 for industrial areas.

The sewer flow hydrograph resulting from the design storm is much different in form to that resulting from a uniform intensity storm. The two hydrographs for storm sewer no. 11 are compared in Figure 13.

The peak sewer flow from the design storm is 37.16 cfs, an increase of 13 per cent over the peak flow from the uniform intensity storm. But, more importantly, the hydrograph has a much flatter recession limb and the total volume of runoff, measured by the area under the hydrograph, is much greater. This discrepancy could be partly overcome by modelling a uniform intensity storm of longer duration so that the total volume of runoff is increased, while the peak flow is decreased only slightly.

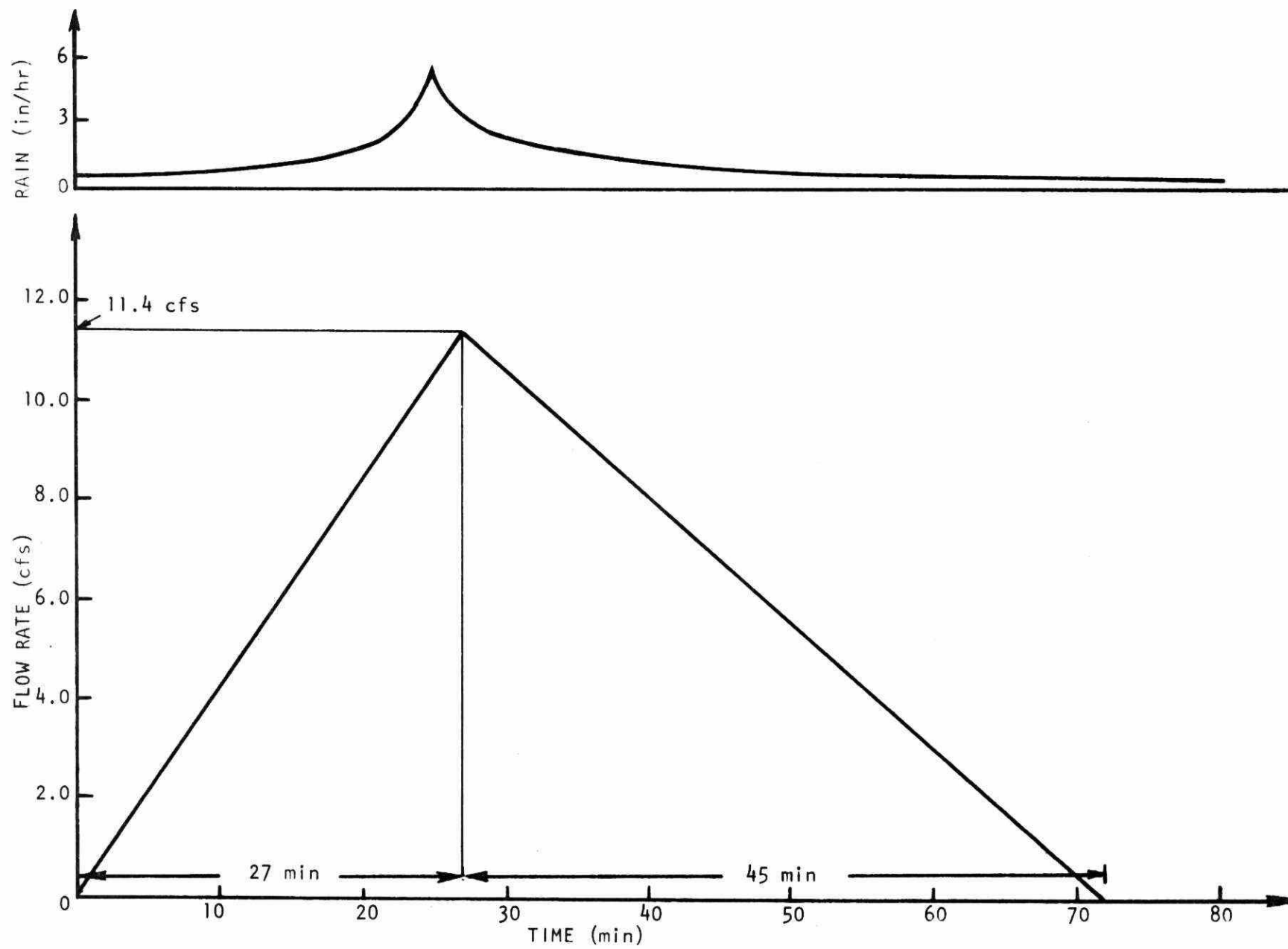


FIGURE 12. IDEALIZED TRIANGULAR RUNOFF HYDROGRAPH FROM INDUSTRIAL AREA A FOR DESIGN STORM

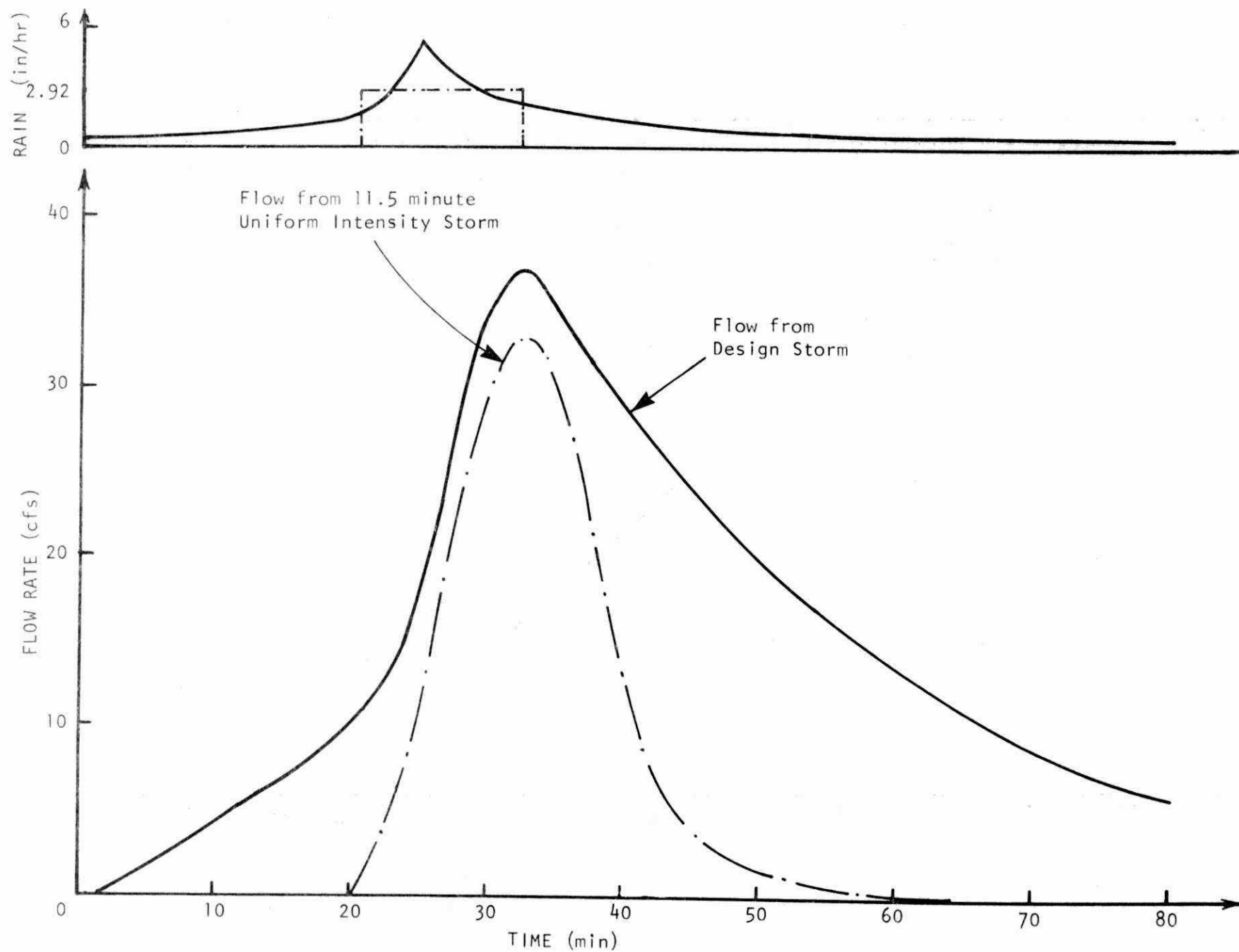


FIGURE 13. COMPARISON OF HYDROGRAPHS IN STORM SEWER NO. 11 FOR 11.5 MINUTE UNIFORM INTENSITY STORM AND FOR DESIGN STORM

It is apparent that the form of the runoff hydrograph is quite sensitive to the choice of design storm. Although the synthetic storm may be considered to have some unreal qualities, it was generally developed from study of a precipitation record and it can constitute an appropriate design storm. An alternative approach could be to simulate actual recorded storms and to use the resulting hydrographs for design purposes.

The hydrograph of flow into a typical catch basin, catch basin no. 21 on storm sewer no. 11, is shown in Figure 14 for the design storm. The peak flow into this catch basin is 0.47 cfs. There are 129 catch basins within the residential part of the subdivision and because of different contributing areas, the hydrographs at each of the catch basins differ. Figure 15 shows the cumulative frequency distribution of peak flows into catch basins. Peak flows vary from 0.12 cfs to 0.94 cfs and the median peak flow is 0.50 cfs and 50 per cent have a peak flow greater than 0.50 cfs.

The peak storm sewer flow estimates from the rational method and from the subdivision hydrograph model, with both uniform intensity storms and the design storm, are compared in Table 6. Good agreement is achieved between the peak flow estimates from the rational method and from the hydrograph model with uniform intensity storms. The corresponding storm durations are from two to four minutes shorter with the hydrograph model. When the design storm is substituted for uniform intensity storms, peak flows are increased by from 10 to 14 per cent.

The hydrograph of storm sewer flow from the subdivision for the design storm is shown in Figure 16. The total flow from the subdivision is the sum of the flows in the storm sewers which join at the outfall, i.e. storm sewers no. 18 and 56. Because of a shorter flow time from the uppermost inlet, the peak flow occurs earlier in storm sewer no. 56. The peak flow from the subdivision is 120.9 cfs and occurs after 34 minutes; the total volume of stormwater outflow after 80 minutes is 264,400 cu ft.

The relative contributions of the residential area, the two industrial areas and the school, and the parks to peak flow and to total stormwater volume are given in Table 7.

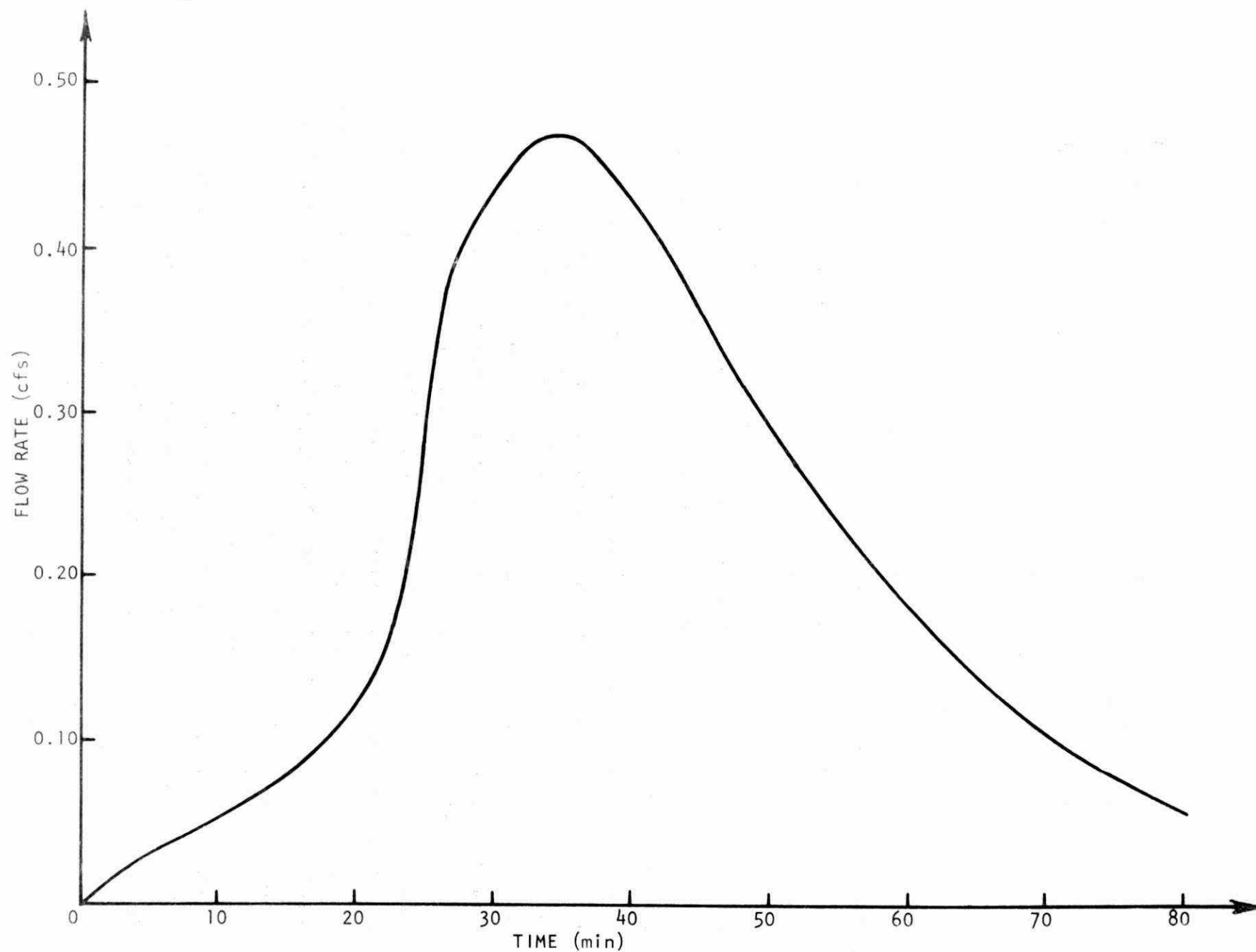


FIGURE 14. HYDROGRAPH OF STORMWATER FLOW INTO TYPICAL CATCH BASIN FOR DESIGN STORM
(CATCH BASIN NO. 21 ON STORM SEWER NO. 11)

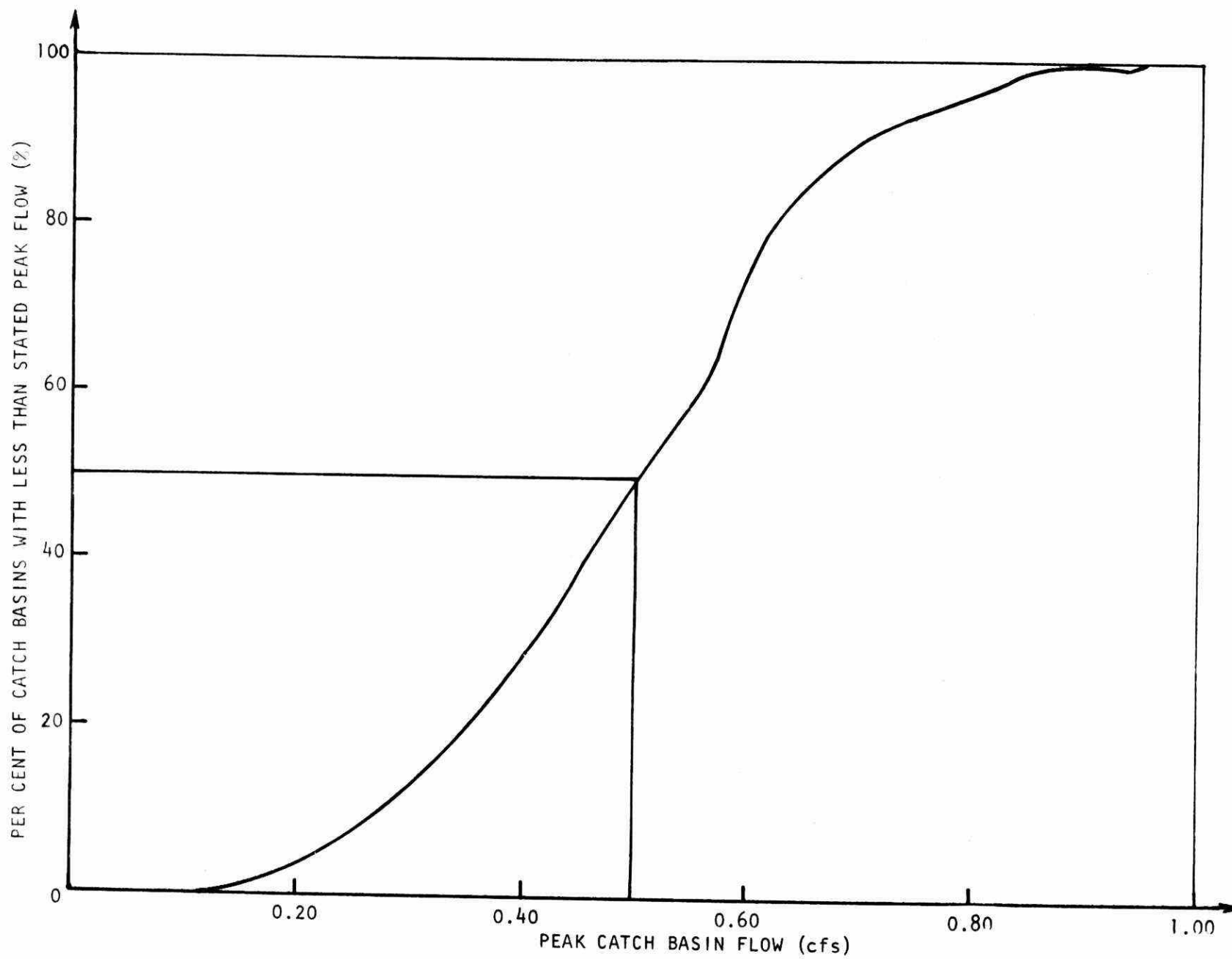


FIGURE 15. CUMULATIVE FREQUENCY DISTRIBUTION OF PEAK FLOWS INTO CATCH BASINS

Table 6
Comparison of Storm Sewer Flow Estimates
from Rational Method and from
Subdivision Hydrograph Model

Storm Sewer No.	Rational Method		Subdivision Hydrograph Model		
			Uniform Intensity Storm		Design Storm
	Storm Duration (min)	Peak Flow (cfs)	Storm Duration (min)	Peak Flow (cfs)	Peak Flow (cfs)
1	7.0	2.50	3.5	2.17	2.41
2	8.3	3.90	4.5	4.37	4.78
3	9.5	8.65	5.5	8.07	8.93
4			6.5	9.47	10.80
5	11.5	17.00	9.0	18.07	19.84
6	12.1	19.40	9.0	18.32	20.92
7	13.1	26.60	9.0	23.80	26.70
8			10.0	24.84	27.94
9	14.3	30.20	10.0	29.20	32.06
10	14.8	33.12	10.0	31.23	34.45
11	15.4	34.61	11.5	32.80	37.16
12	16.4	48.10	12.0	44.02	49.98
13	17.3	47.83	12.0	44.72	51.04
14	17.8	67.96	13.0	68.11	73.82
15	18.2	69.23	14.5	69.73	76.99
16	19.3	69.63	15.5	70.28	79.02
17	20.4	80.40	17.5	81.02	91.61
18	20.7	89.39	17.5	87.34	99.58

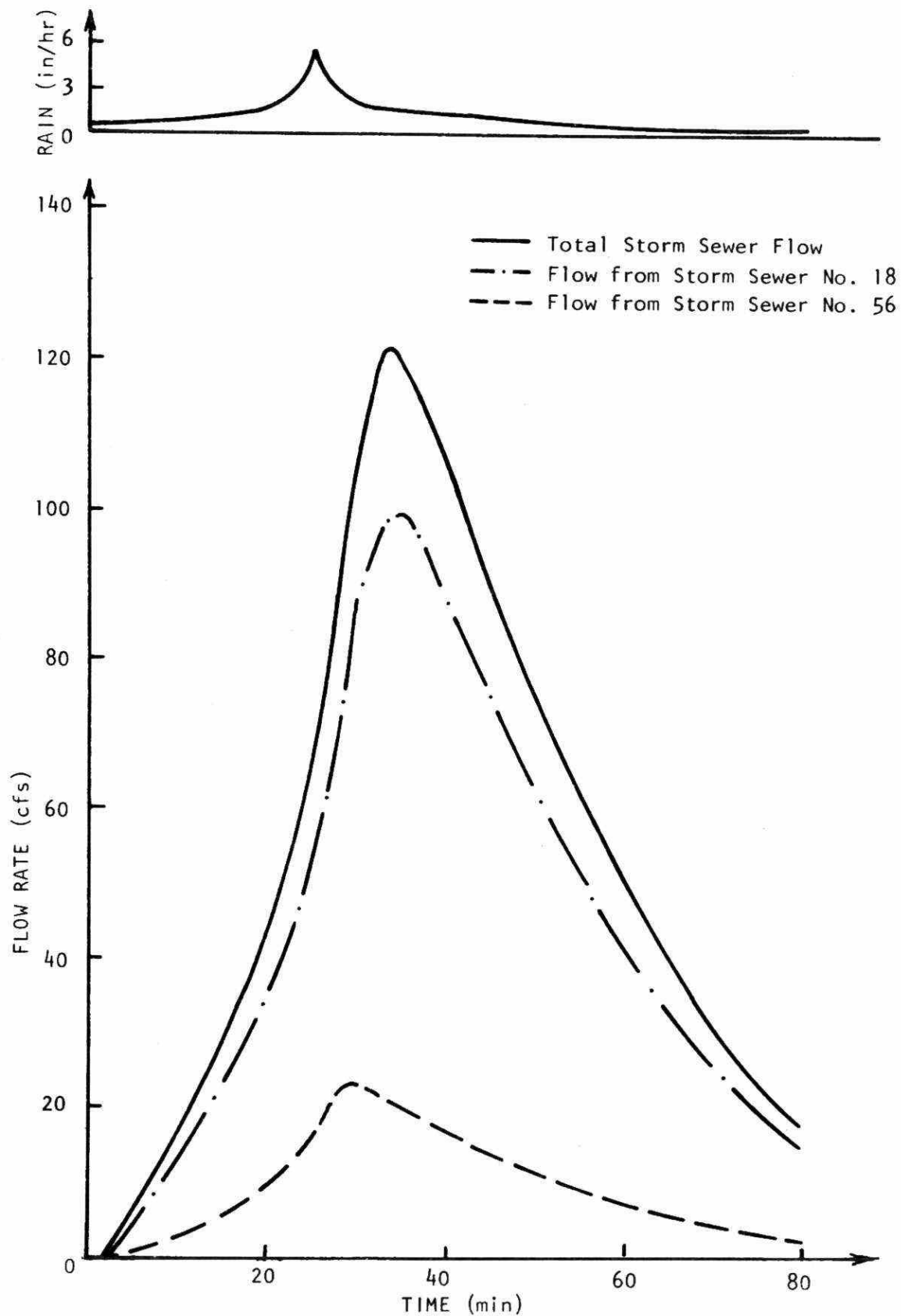


FIGURE 16. HYDROGRAPH OF STORM SEWER FLOW FROM SUBDIVISION FOR DESIGN STORM

Table 7
Contributions of Different Land Uses
to Storm Sewer Flows from Subdivision

Land Use	Area (acres)	Peak Flow (cfs)	Peak Flow/ Unit Area (cfs/acre)	Stormwater Volume (cu ft)
Residential	83.2	94.7	1.14	202,500
Industrial, School	10.5	21.2	2.02	50,000
Park	7.9	5.0	0.63	11,900
Total	101.6	120.9	1.19	264,400

The results indicate that, although they occupy only a small part of the catchment, the industrial areas and the school make a relatively large contribution to stormwater flows because of their high imperviousness. The higher cfs/acre value suggests that the benefits of storage could be greater in these areas and that they should be subject to detailed study. However, as the purpose of this study was to investigate the effects of storage in residential areas, simplified triangular hydrographs of the form shown in Figure 12 were retained for non-residential areas.

4. COMBINED SEWER DESIGN AND FLOW ESTIMATION

4.1 Existing Sanitary Sewer System

Design of the existing sanitary sewer system for the subdivision was based on the following peak wastewater flow rates:

	<u>maximum hourly flow</u>
residential area	.015 cfs/acre
industrial area, schools	.025 cfs/acre

As well as flows from the land uses within the 101.6 acre subdivision, the sanitary sewer system was designed to accommodate a peak inflow of 2.0 cfs from an 80 acre industrial area at the upstream end of sanitary sewer no. 1. Pipe diameters in the main line of the sanitary network are 15 inches for sewers no. 1 to 13 and 18 inches for sewers no. 14 to 18. The diameter of all other links in the network is eight inches, the minimum pipe size allowed by the municipality.

4.2 Combined Sewer Design

For combined sewer design, the two-year design storm was assumed to be coincident with the peak sanitary flow and the sewers were sized to handle the combined sanitary and stormwater flows.

The principles of combined sewer design are similar to those of storm sewer design except that higher minimum velocities are desirable. To provide adequate movement of dry weather flows, a minimum sewer full-flow velocity of five ft/sec is recommended (19). This means that combined sewers must be laid at steeper grades than storm sewers. Also with higher full-flow velocities, manhole losses are greater for combined sewers. The allowances for losses were increased from 0.10 feet (with storm sewers) to 0.20 feet for in-line manholes, and from 0.25 feet to 0.35 feet for manholes where a change in pipe size occurred. An allowance of 0.50 feet was made for junction manholes.

A combined sewer design was carried out and details of peak flows, sewer diameters and full-flow velocities, and average depths are presented in Table 8 for the main sewer line.

Table 8

Details of Combined Sewer Design for Subdivision

Combined Sewer No.	Peak Flow (cfs)	Diameter (in)	Full-Flow Velocity (ft/sec)	Average Depth (ft)
1	4.56	15	5.1	13.9
2	7.04	18	5.1	14.9
3	11.32	21	5.1	14.8
4	13.18	24	5.6	16.7
5	22.74	30	5.6	18.7
6	23.96	30	5.3	21.2
7	30.17	33	5.3	21.8
8	31.40	36	5.3	22.1
9	35.69	36	5.2	22.1
10	38.35	39	5.5	21.8
11	41.13	39	5.5	20.7
12	54.40	48	5.7	20.9
13	55.48	48	5.7	22.6
14	78.84	54	5.5	24.7
15	82.02	54	5.5	26.9
16	84.05	54	5.5	27.3
17	96.70	60	5.9	24.7
18	104.67	60	5.9	24.7

The hydrograph model was re-run with the sanitary flows included and the peak sewer flows were obtained from the corresponding hydrographs. The combined sewer diameters are, at most, one size increment larger than the separate storm sewer diameters. The time of flow through the combined sewer network is 12.2 minutes because of higher full-flow velocities. The average depth of sewers is increased considerably, and the depth of a combined sewer at the outfall from the subdivision would be about six feet greater than that of the existing storm sewer.

The increased depth of combined sewers follows directly from the requirement of a higher full-flow velocity than for storm sewers. If velocities less than five ft/sec are accepted for combined sewers, average depths will be reduced and the costs of the combined sewer system will be lowered. The sensitivity of sewer costs to changes in full-flow velocity was not tested in this research.

Since cost functions relating unit sewers costs to sewer diameters and depths were not considered at the design stage, the degree of optimality reached in the combined sewer design cannot be stated precisely. However, high excavation costs generally dictate that sewers be located as near as practicable to the surface, within the constraints of minimum acceptable sewer depths and velocities. This consideration was included in the combined sewer design.

4.3 Combined Sewer Flows from Subdivision

The hydrograph of combined sewer flow from the subdivision for the design storm is shown in Figure 17. The relative contributions of sanitary flow and stormwater flow are as follows:

	<u>Peak Flow (cfs)</u>	<u>Total Flow Volume (cu ft)</u>
Sanitary	3.4	16,300
Stormwater	122.3	264,900
<u>Total</u>	<u>125.7</u>	<u>281,200</u>

The peak combined flow for the design storm is about 40 times the peak sanitary flow. For investigation of low flow conditions, the following ratios were assumed for sanitary flows:

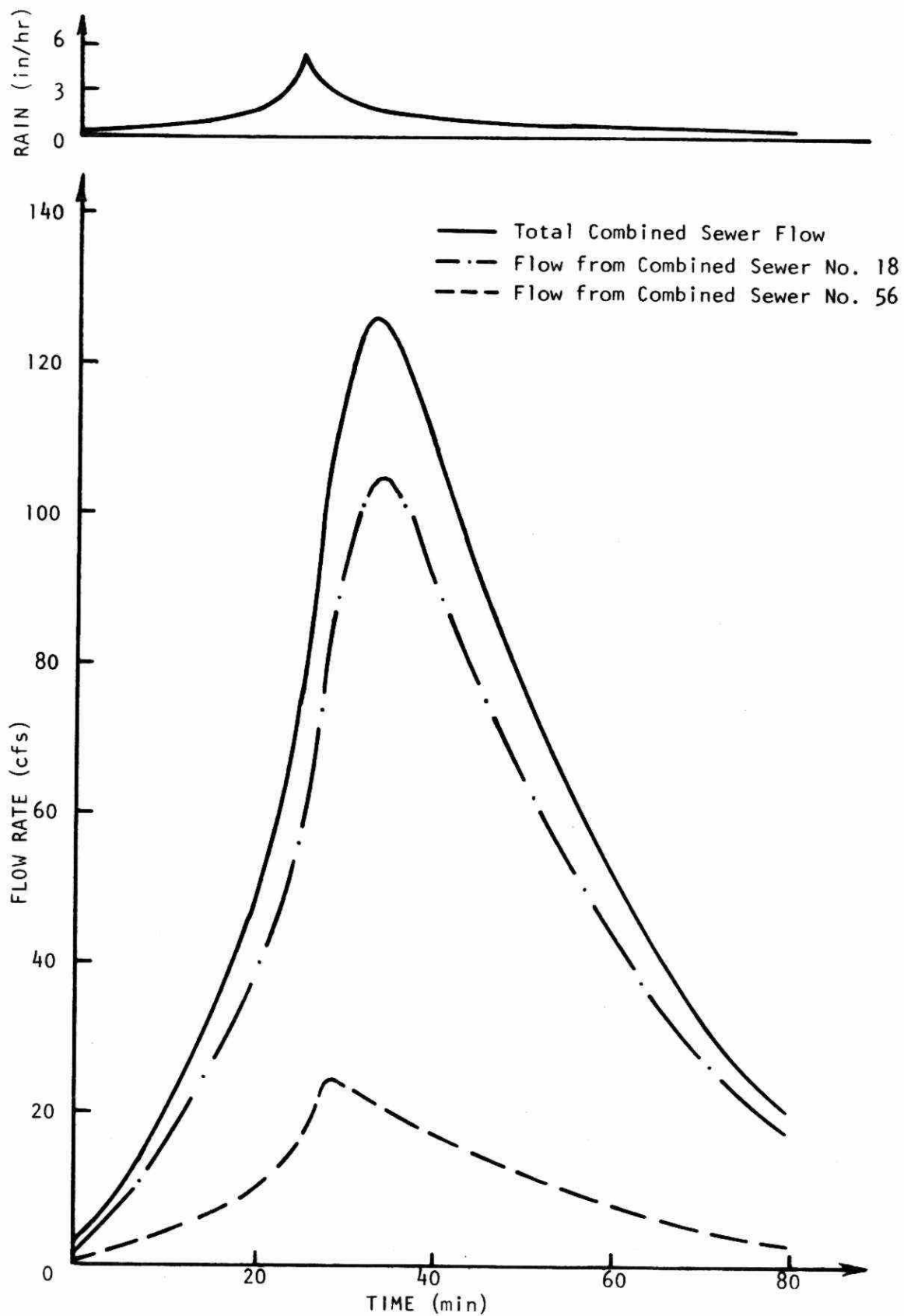


FIGURE 17. HYDROGRAPH OF COMBINED SEWER FLOW FROM SUBDIVISION FOR DESIGN STORM

$$\frac{\text{maximum hourly flow}}{\text{average daily flow}} = \frac{3}{1}$$

$$\frac{\text{minimum hourly flow}}{\text{average daily flow}} = \frac{1}{3}$$

Hence the peak wet weather combined flow is about 120 times the average dry weather sanitary flow.

The velocities of sanitary flows in the combined sewers during dry weather periods would be less than those in separate sanitary sewers. For example, the peak flow in sanitary sewer no. 18 is 3.07 cfs. From the above ratios, the minimum hourly flow

$$= 3.07 \times 1/3 \times 1/3 = 0.34 \text{ cfs}$$

The flow velocity at the minimum hourly flow rate in the 18 inch diameter sewer is 1.8 ft/sec. The flow velocity in a 60 inch combined sewer would be 1.3 ft/sec, so that there would be a greater tendency for solids to accumulate in the combined sewer during dry weather periods.

5. EFFECTS OF STORAGE

5.1 Storage Routing Functions

This section describes the routing functions incorporated in the model for each of the storage methods. The fundamental relationship involving storage is:

$$\text{rate of inflow} = \text{rate of outflow} + \text{rate of storage}$$

The rate of outflow is governed by the routing function adopted.

5.1.1 Roof ponding

Rainfall may be detained on flat roofs by means, for example, of raised outlets. Runoff from a roof is zero until the depth of rain exceeds the detention depth, after which the overflow becomes the roof runoff hydrograph. Figure 18 illustrates the effect of allowing one inch of ponding on the runoff hydrograph from a typical 1700 sq ft roof for the two-year design storm. Runoff commences after 30 minutes and the total volume of water stored on the roof is 142 cu ft.

5.1.2 Site storage of roof runoff

When ponding on roofs may not be an acceptable practice, roof runoff may be conveyed to a site storage facility such as an underground tank from which the runoff is released to the sewer at a controlled rate. For preliminary analysis, a constant outflow from the site storage was assumed. A constant outflow has previously been used in the investigation of local storage alternatives for Denver (24). The effect of allowing a variable release rate from storage was not tested in this research, but alternative methods of storage operation would need to be considered at the design stage.

The effect of a constant outflow from site storage of 0.04 cfs on the roof runoff hydrograph is shown in Figure 19. The storage begins to fill after 15 minutes and to empty after about 40 minutes. The required volume is 63 cu ft.

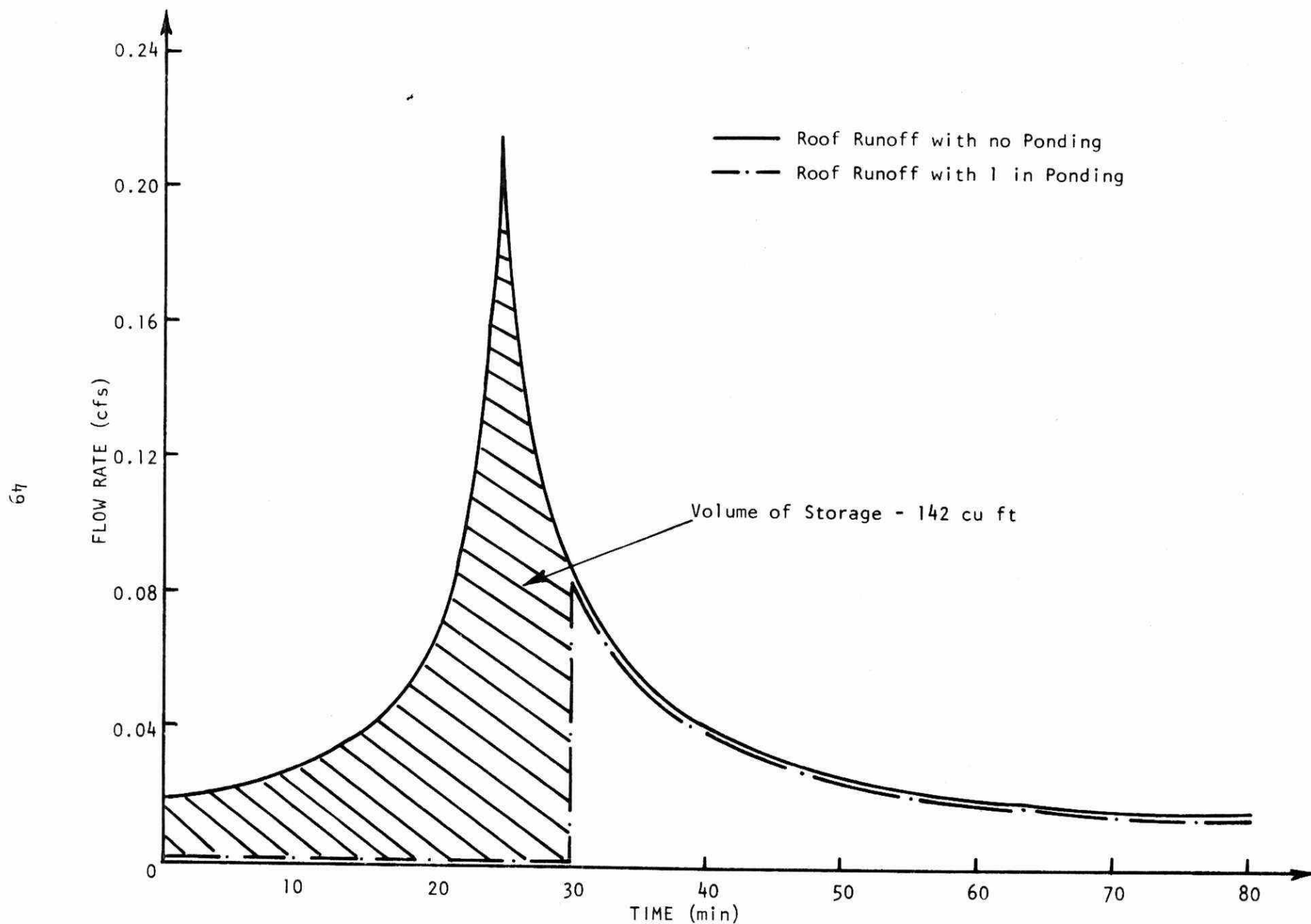


FIGURE 18. EFFECT OF ROOF PONDING ON ROOF RUNOFF HYDROGRAPH FOR TYPICAL 1700 sq ft ROOF

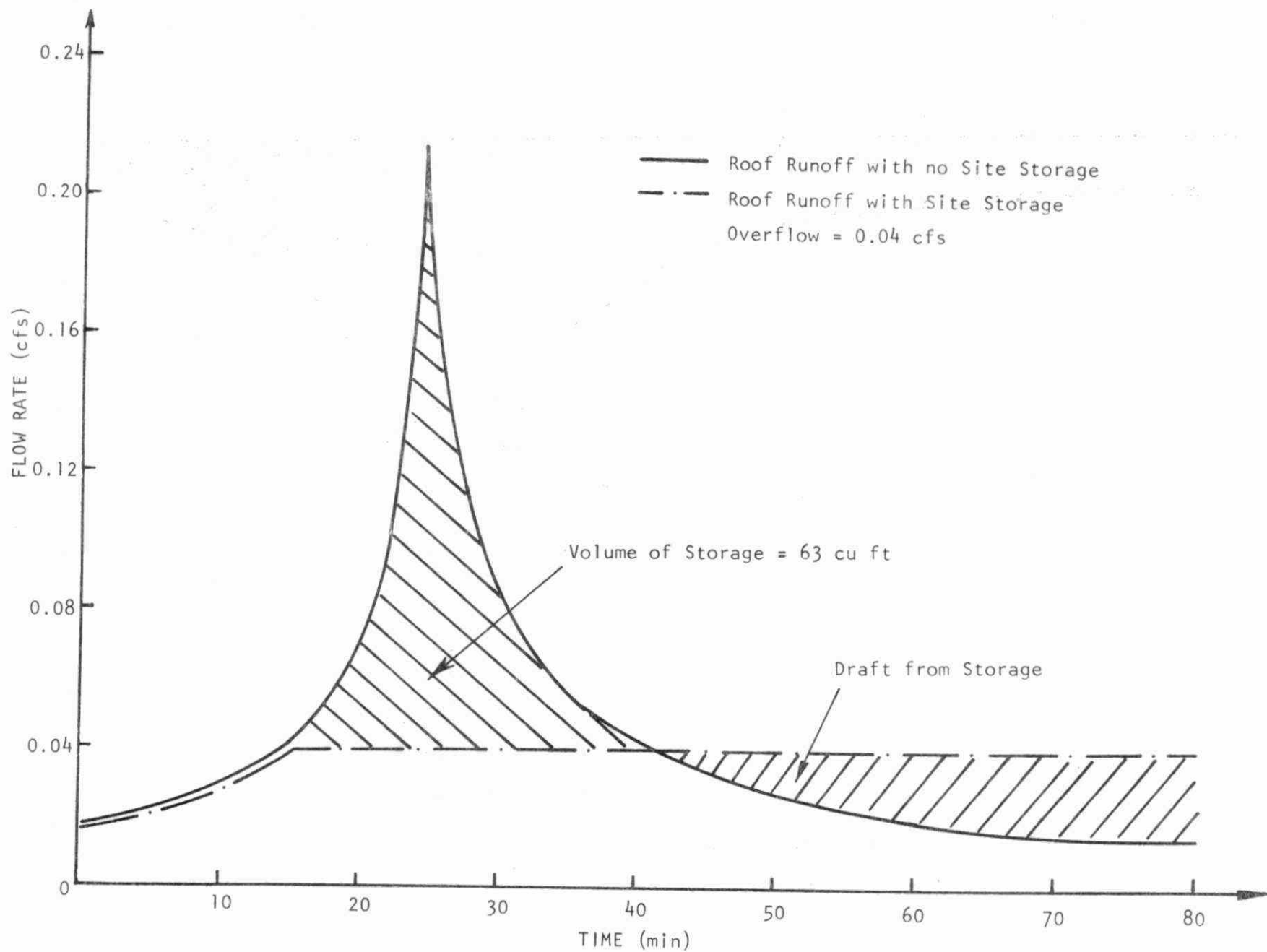


FIGURE 19. EFFECT OF SITE STORAGE ON ROOF RUNOFF HYDROGRAPH FOR SINGLE RESIDENTIAL LOT

5.1.3 Catch basin storage

In conventional sewer systems, the purpose of catch basins is to remove grit which might otherwise be deposited in sewers. It would seem worthwhile to utilize the volumetric capacity of catch basins or of similar underground structures to reduce peak flows from street gutters into sewers. Figure 20 illustrates the effect of limiting outflow to 0.30 cfs on the hydrograph for catch basin no. 21. The storage volume needed at this catch basin is 158 cu ft.

Because of the different flows to each of the 129 catch basins in the subdivision, the storage requirements for a given outflow vary. The cumulative frequency distribution of storage volumes for an outflow of 0.30 cfs is shown in Figure 21. Eleven per cent of the catch basins have peak inflows less than 0.30 cfs and require no storage. The maximum storage requirement is 800 cu ft, and the median storage volume is about 180 cu ft.

5.1.4 Holding reservoirs

Holding reservoirs may be located at various points in the sewer network to modify sewer flow-time patterns during storms. A constant outflow could be assumed for a holding reservoir. In practice, this could be achieved by constant rate pumping from the reservoir to a downstream sewer.

If control by gravity is desired, outflow from the reservoir could probably be governed by an orifice or weir. A stepwise procedure similar to that for calculating overland flow has been adopted for the case of orifice control. The steps in the procedure are as follows:

1. The depth of water after inflow occurs is calculated:

$$Y_1 = Y_t + \frac{Q_{in}}{A_h} \Delta t$$

where: Y_1 is water depth after inflow occurs (ft);
 Y_t is water depth at time t (ft);
 Q_{in} is inflow in time interval Δt (cfs);
 A_h is area of holding reservoir (sq ft);
 Δt is length of time interval (sec).

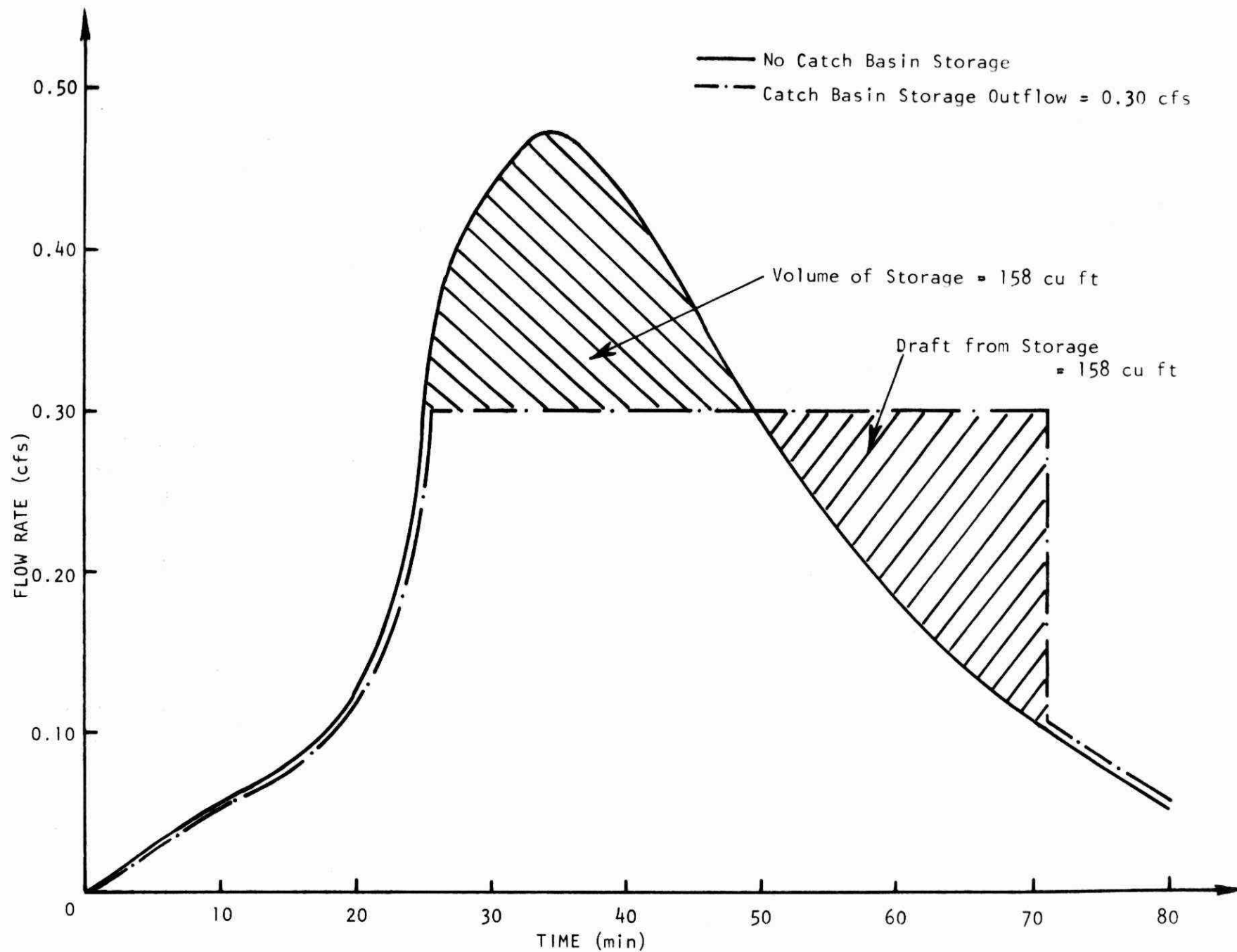


FIGURE 20. EFFECT OF STORAGE ON TYPICAL CATCH BASIN OUTFLOW HYDROGRAPH (CATCH BASIN NO. 21 ON STORM SEWER NO. 11)

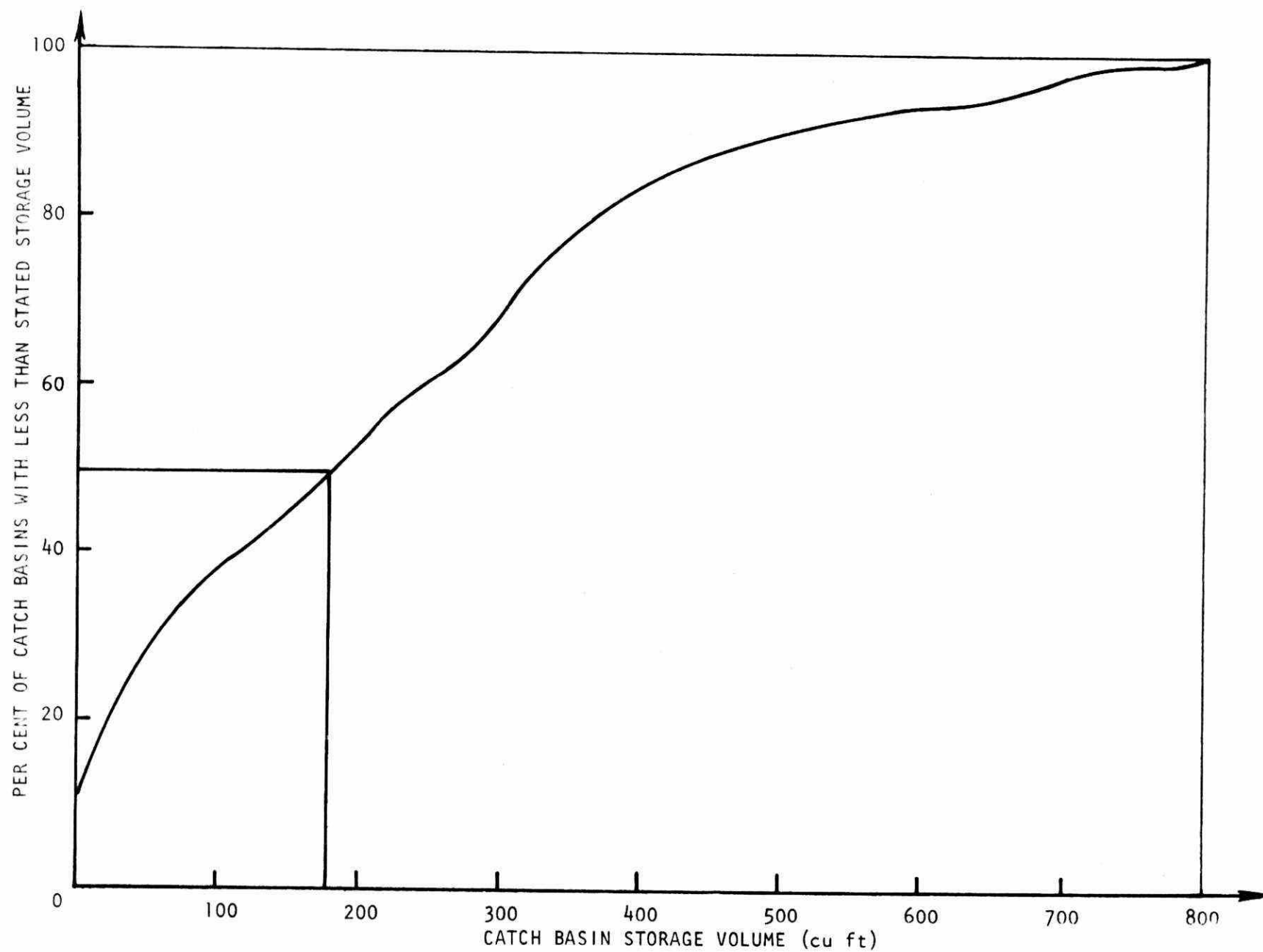


FIGURE 21. CUMULATIVE FREQUENCY DISTRIBUTION OF CATCH BASIN STORAGE VOLUMES FOR STORAGE OUTFLOW = 0.30 cfs

2. The outflow rate is calculated using the orifice formula:

$$Q_{out} = 0.62 \times A_o \times \sqrt{2g H_o}$$

where: Q_{out} is outflow in time interval Δt (cfs);
 A_o is area of orifice (sq ft);
 H_o is head of water above orifice centreline (ft)
 $= Y_1 - \text{orifice diameter}/2.0$

3. The depth of water at the end of the time interval is calculated:

$$Y_t + \Delta t = Y_1 - \frac{Q_{out}}{A_h} \Delta t$$

4. Steps 1 to 3 are repeated for successive time intervals.

5. The required storage volume is calculated

$$\text{Storage volume} = Y_{1 \max} \times A_h$$

where: $Y_{1 \max}$ is maximum value of Y_1

5.2 Effects of Storage on Storm Sewer Flows

The effect of each storage type on the subdivision outflow hydrograph from the design storm was investigated for a range of storage parameter values. The results for separate storm sewers with eight storage options are summarized in Table 9. For each option, the peak flow and the volume of outflow during the 80 minute simulation period are recorded. The reduction in peak flow rate is a primary measure of the effectiveness of storage. The reduction in outflow volume measures the effectiveness of storage in limiting runoff during the rainfall period, when demands on the wastewater collection system are greatest. The storage methods do not reduce the total volumes of runoff.

The unit storage volume shown in Table 9 is the volume required for the appropriate storage facility. For catch basin storage, which varies between individual catch basins, the maximum storage requirements have been recorded.

For roof ponding (option 1), site storage of roof runoff (option 2) and catch basin storage (option 3), cases (a), (b), (c) and (d) assume no storage of runoff from industrial areas. Case (e)

Table 9

Effects of Storage on Storm Sewer Flows from Subdivision

Storage Option	Unit Storage Volume (cu ft)	Peak Outflow (cfs)	Reduction in Peak Outflow (%)	Outflow Volume** (cu ft)	Reduction in Outflow Volume(%)
No Storage	0	120.9	---	264,400	---
1. <u>Roof ponding</u>					
Ponding depth (a) 0.5 in	71	119.7	1.0	248,400	6.1
(b) 1.0 in	142	99.5	17.7	221,600	16.2
(c) 1.5 in	213	82.7	31.5	193,600	26.8
(d) 2.0 in	260	82.7	31.5	189,700	28.2
(e) 2.0 in*	260	75.0	37.9	169,700	35.8
2. <u>Site storage of roof runoff</u>					
Outflow rate (a) 0.08 cfs	24	113.3	6.3	264,000	0.2
(b) 0.04 cfs	63	98.0	18.9	252,800	4.4
(c) 0.02 cfs	109	90.3	25.3	223,600	15.4
(d) 0.01 cfs	158	86.5	28.4	206,700	21.8
(e) 0.01 cfs*	158	78.8	35.7	186,700	29.4
3. <u>Catch basin storage</u>					
Outflow rate (a) 0.40 cfs	520	111.3	7.9	260,200	1.6
(b) 0.30 cfs	800	104.2	13.8	248,200	5.9
(c) 0.20 cfs	1,100	94.9	21.5	223,100	15.6
(d) 0.10 cfs	1,500	83.7	30.7	186,600	29.5
(e) 0.10 cfs*	1,500	74.9	38.0	166,600	37.0
4. <u>Holding reservoir at outfall (constant outflow)</u>					
Outflow rate (a) 90.0 cfs	20,500	90.0	25.6	264,400	0.0
(b) 60.0 cfs	64,600	60.0	50.2	236,600	10.6
(c) 30.0 cfs	140,400	30.0	75.0	128,500	51.4

Continued...../

Table 9 (cont'd)

Effects of Storage on Storm Sewer Flows from Subdivision

Storage Option	Unit Storage Volume (cu ft)	Peak Outflow (cfs)	Reduction in Peak Outflow (%)	Outflow Volume** (cu ft)	Reduction in Outflow Volume(%)
<u>5. Holding reservoir at outfall (orifice outflow control)</u>					
Reservoir area= 10,000 sq ft					
Orifice diameter (a) 18 in	164,300	35.0	71.0	106,700	59.8
(b) 30 in	95,200	70.3	41.7	210,800	20.3
Reservoir area = 20,000 sq ft					
Orifice diameter (c) 18 in	191,200	26.1	78.2	76,000	71.1
(d) 30 in	129,000	55.7	53.9	163,000	38.4
<u>6. Roof ponding and catch basin storage</u>					
Ponding depth = 2 in	260				
Catch basin outflow rate = 0.30 cfs	800	61.1	49.4	173,500	34.4
<u>7. Holding reservoir at upstream manhole of sewer no. 14</u>					
Outflow rate = 50.0 cfs	15,600	105.8	12.5	264,400	0.0
<u>8. Holding reservoirs both upstream and at outfall</u>					
Outflow rates:					
upstream 50.0 cfs	15,600				
at outfall 60.0 cfs	56,200	60.0	50.2	236,600	10.6

* Industrial area runoff reduced by 40%.

** Outflow volumes for 80 minutes duration.

considers a 40 per cent reduction in industrial area runoff. This figure was established by examining the flow reductions for a purely residential area, namely the 29.7 acre area contributing to storm sewer no. 11. The effects of storage options 1(d), 2(d), and 3(d) on flows in storm sewer no. 11 are shown in Table 10. Flow reductions of about 40 per cent occur in each case and it seems reasonable to allow a similar reduction for industrial areas, if the same storage is incorporated in them.

5.2.1 Roof ponding

A ponding depth of 0.5 inches is not effective in reducing peak flow from the subdivision for the design storm because overflow from the roof takes place before the peak rainfall intensity occurs. As ponding depth is increased, peak flow is reduced, with a 31.5 per cent reduction for a ponding depth of two inches. Similar levels of reduction in volumes of outflow are achieved. In fact, allowance for ponding greater than the total rainfall of 1.84 inches will not decrease outflows any further because all roof runoff has been eliminated. The total volume of water stored on a typical 1,700 sq ft roof is 260 cu ft. When additional storage is provided in industrial areas, peak flow is reduced by a further 6.4 per cent. Figure 22 shows that the effect of roof ponding is to lower the subdivision outflow hydrograph.

5.2.2 Site storage of roof runoff

Site storage of roof runoff modifies the steep portions of the rising and recession limbs of the roof runoff hydrograph (Figure 19). For storage outflow rates between 0.08 cfs and 0.01 cfs, reductions in peak flows from the subdivision are of similar magnitude to those for roof ponding. But because runoff is released from site storage with little delay, the reductions in stormwater volumes from the subdivision over the 80 minute period are not as great. Depending on the storage outflow rate, the storage volume required per residential lot varies from 24 cu ft to 158 cu ft. The effect of site storage of roof runoff is to lower the outflow hydrograph peak and to flatten the recession limb, as shown in Figure 23.

Table 10

Effects of Storage on Flows in Storm Sewer No. 11

Storage Option	Peak Sewer Flow (cfs)	Reduction in Peak Sewer Flow (%)	Flow Volume (cu ft)	Reduction in Flow Volume (%)
No Storage	37.16	---	72,900	--
1. <u>Roof ponding</u> Ponding depth = 2 in	21.40	42.4	45,900	37.0
2. <u>Site storage of roof runoff</u> Outflow rate = 0.01 cfs	22.78	38.7	52,100	28.5
3. <u>Catch basin storage</u> Outflow rate = 0.10 cfs	22.42	39.6	45,600	37.5

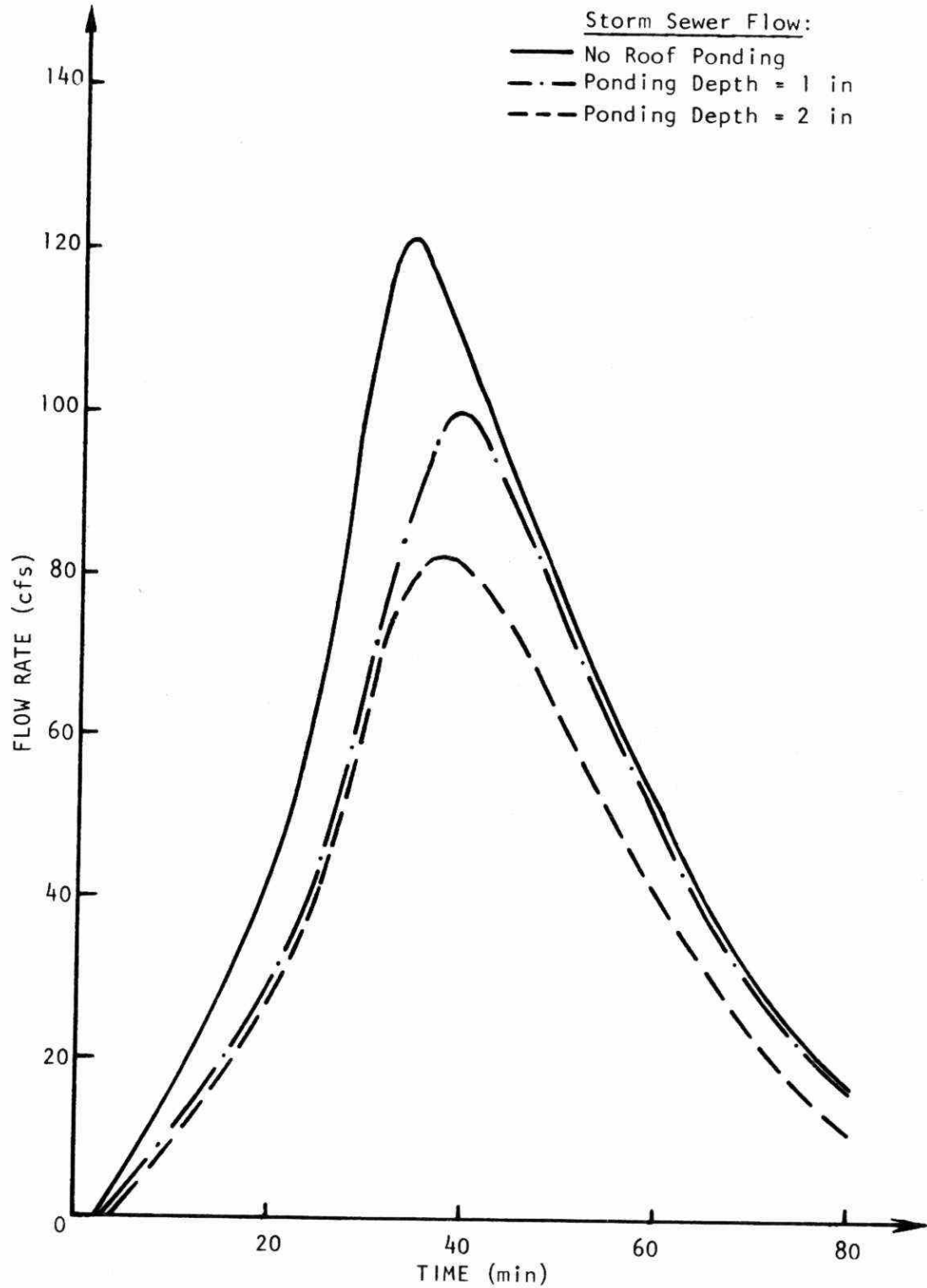


FIGURE 22. EFFECT OF ROOF PONDING ON STORM SEWER FLOW FROM SUBDIVISION

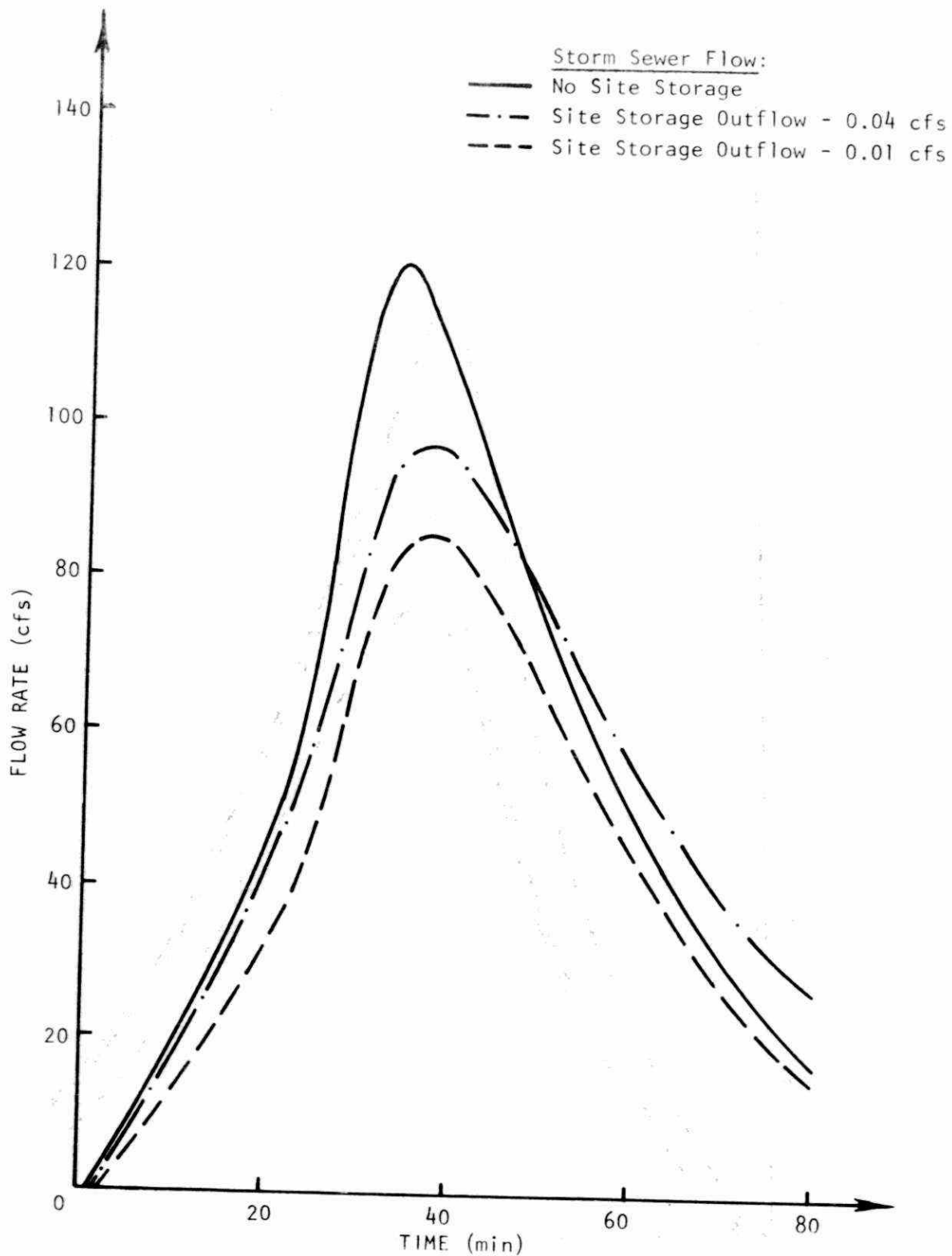


FIGURE 23. EFFECT OF SITE STORAGE OF ROOF RUNOFF ON STORM SEWER FLOW FROM SUBDIVISION

5.2.3 Catch basin storage

As the outflow rate from each catch basin is lowered from 0.40 cfs to 0.10 cfs, the reduction in peak storm sewer flow from the subdivision increases from 7.9 per cent to 30.7 per cent, and to 38.0 per cent with additional storage in industrial areas. For a given storage outflow rate, the required storage volume varies between catch basins. The storage required at the catch basin with the largest inflow increases from 520 cu ft to 1,500 cu ft as the outflow rate is lowered. The effect of catch basin storage on the subdivision outflow hydrograph is illustrated in Figure 24. Because the directly connected roof runoff becomes the dominant contributor to sewer flow, the peak outflow is slightly advanced in time.

5.2.4 Holding reservoir at outfall (constant outflow)

When a holding reservoir is located at the outfall, the opportunity exists for centralized large-scale modification of flows from the subdivision. The outflow rates can be made as small as desired, but corresponding increases in storage volumes result. For example, when pumping from the reservoir is provided, the pumping rate can be fixed at 90 cfs and 20,500 cu ft of storage is necessary. The form of the outflow hydrograph for a holding reservoir with constant outflow is shown in Figure 25.

5.2.5 Holding reservoir at outfall (orifice outflow control)

In a gravity flow system, the size of an orifice outlet can be adjusted to modify flow from storage. For example, if an area of 10,000 sq ft is available for a holding reservoir, a 30 inch diameter orifice will limit the peak outflow to 70.3 cfs. The storage volume needed is 95,200 cu ft. As well as reducing peak flow, orifice control increases the time to peak, as shown in Figure 26.

5.2.6 Roof ponding and catch basin storage in combination

The storage options may be used in combination as well as separately. If roof runoff is eliminated by ponding to a depth of two

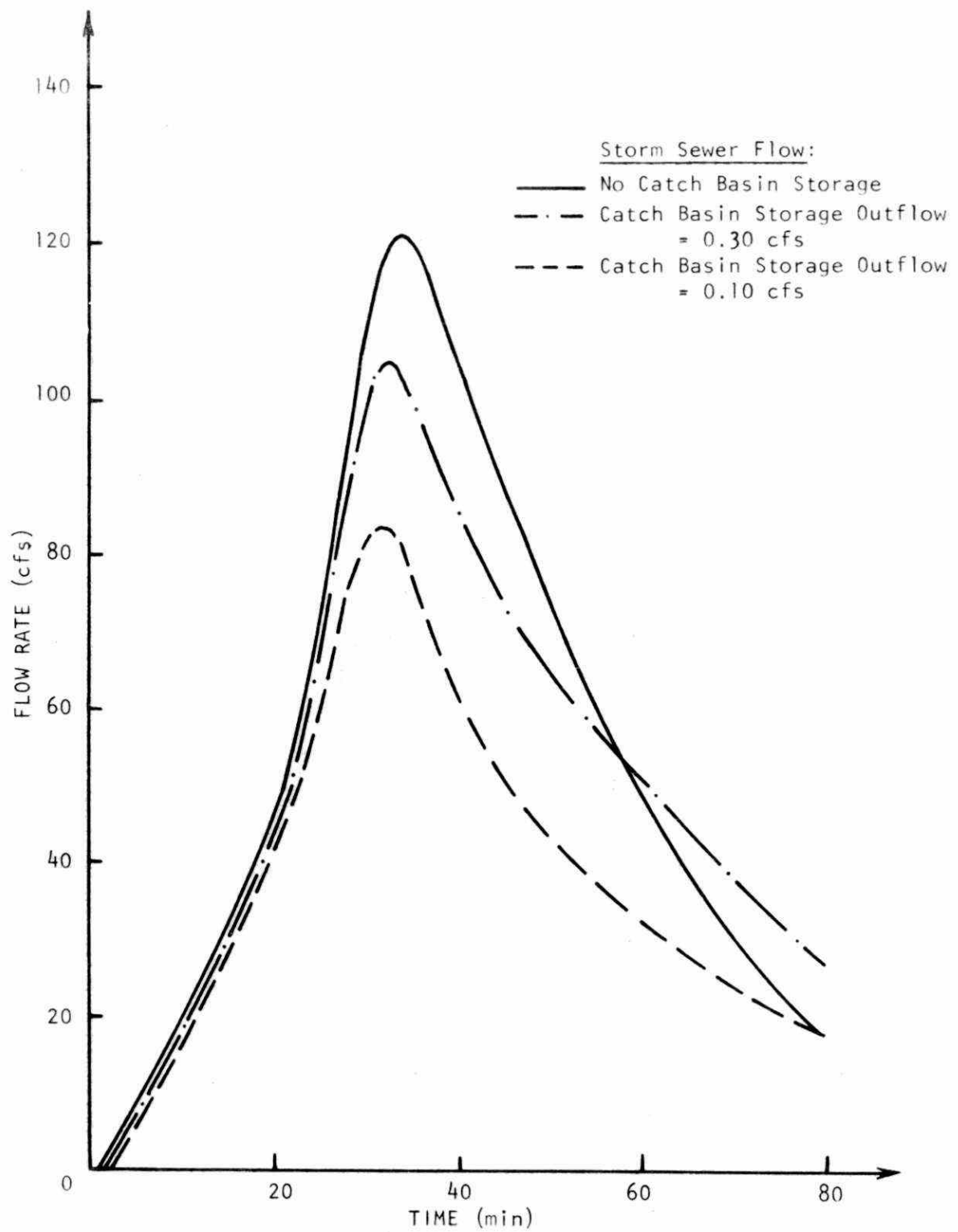


FIGURE 24. EFFECT OF CATCH BASIN STORAGE ON STORM SEWER FLOW FROM SUBDIVISION

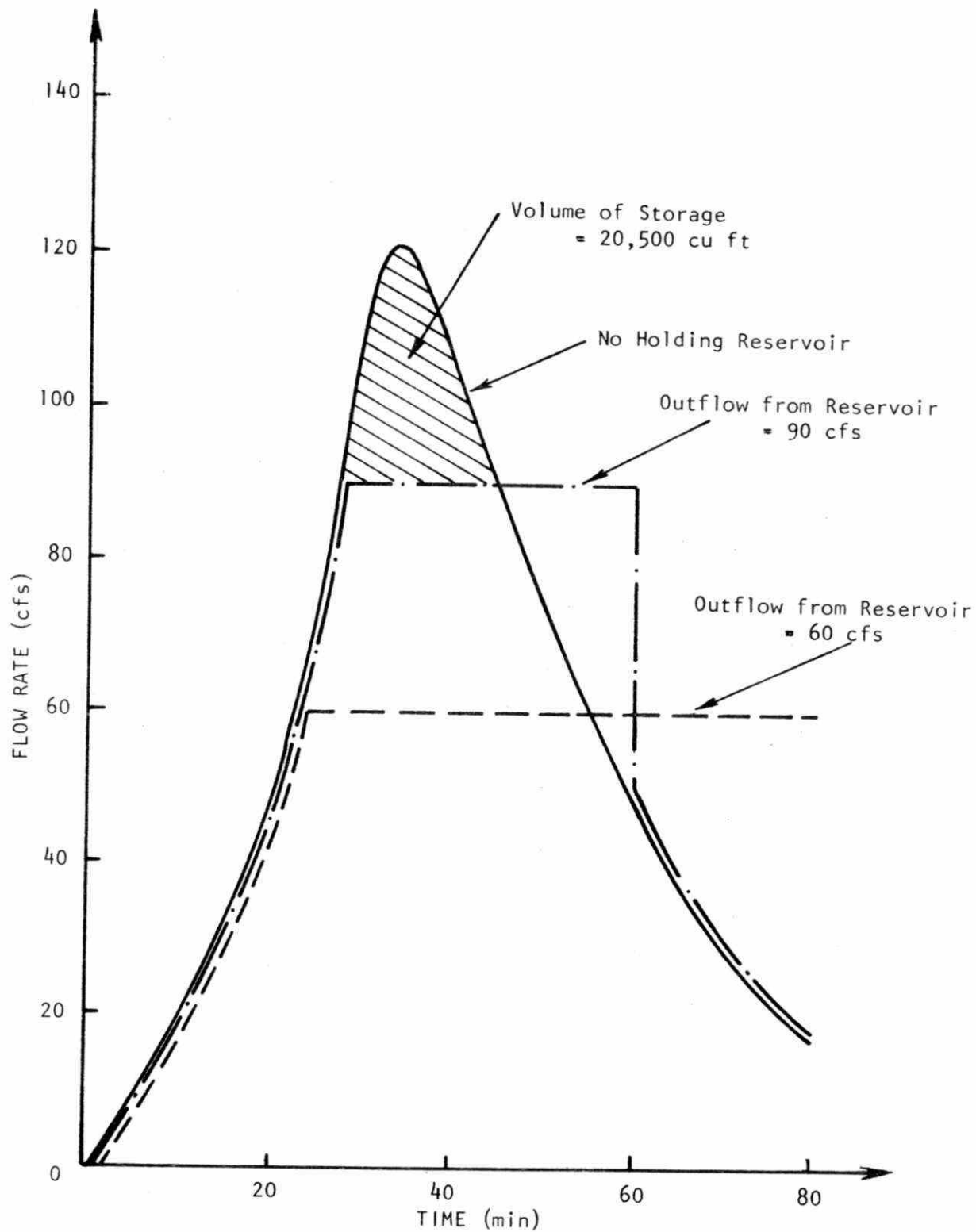


FIGURE 25. EFFECT OF HOLDING RESERVOIR WITH CONSTANT OUTFLOW ON STORM SEWER FLOW FROM SUBDIVISION

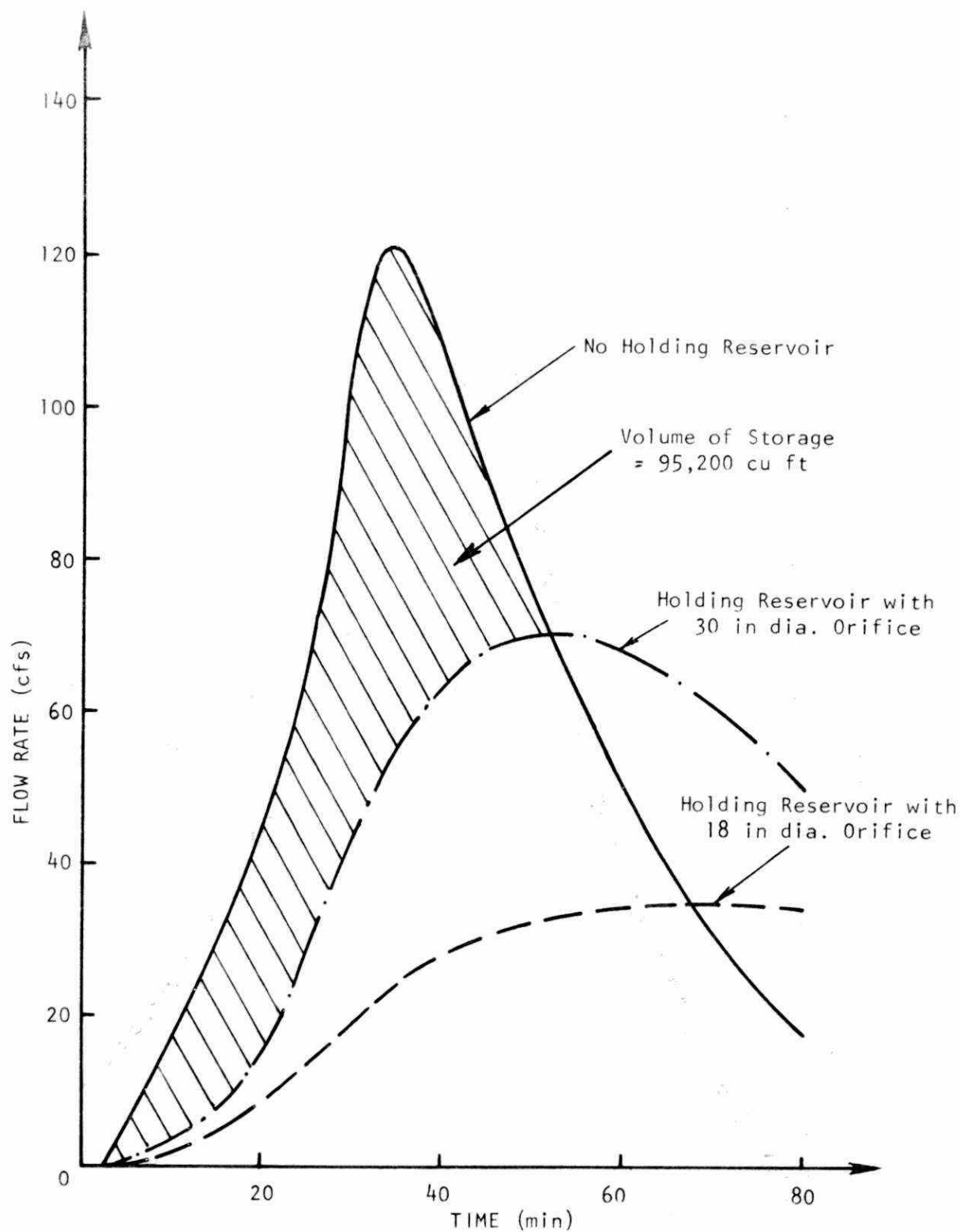


FIGURE 26. EFFECT OF HOLDING RESERVOIR WITH ORIFICE OUTFLOW CONTROL ON STORM SEWER OUTFLOW FROM SUBDIVISION (Reservoir Area = 10,000 sq ft)

inches and if outflow from catch basins is restricted to 0.30 cfs, peak outflow from the subdivision is reduced by almost 50 per cent.

5.2.7 Holding reservoir at upstream locations

The effects of locating a holding reservoir upstream from the outfall were also investigated. Flow modification at upstream points may enable reduction in downstream sewer sizes. Two major sewer lines join at the upstream manhole of sewer no. 14. Without storage, the peak inflow to storm sewer no. 14 is 72.8 cfs. If the inflow is limited to 50 cfs by means of a small holding storage, the peak flows in sewers no. 14 to 18 are lowered and the peak outflow from the subdivision is reduced from 120.9 cfs to 105.8 cfs. The required storage volume is 15,600 cu ft.

5.2.8 Holding reservoirs at upstream location and at outfall

Holding reservoirs may be located at various points in the sewer network. If storages are placed both at the upstream manhole of storm sewer no. 14 and at the outfall, the peak flow from the subdivision may be reduced to 60.0 cfs with a total storage volume of (15,600 + 56,200) 71,800 cu ft. The same reduction in peak flow would be achieved by a single reservoir at the outfall with a storage volume of 64,600 cu ft.

5.3 Effects of Storage on Combined Sewer Flows

The effects of storage on combined sewer flows from the subdivision were also investigated, and the results are summarized in Table 11. Storage options 7 and 8, involving on-line holding reservoirs within the subdivision, were not tested for the combined sewer network. Water quality problems associated with combined sewer flows would probably make the use of these options impractical for very small drainage areas. These problems might be overcome to some extent by providing off-line storage, which would be utilized only during heavy storms.

The results are quite similar to those for separate storm sewers. For roof ponding, site storage of roof runoff, and catch basin storage, all of which would involve detention of stormwater flows, the storage requirements are unchanged. But because of the inclusion of sanitary flows, the per cent reductions in subdivision outflows are

Table 11

Effects of Storage on Combined Sewer Flows from Subdivision

Storage Option	Unit Storage Volume (cu ft)	Peak Outflow (cfs)	Reduction in Peak Outflow(%)	Outflow Volume (cu ft)	Reduction in Outflow Volume(%)
No Storage	0	125.7		281,200	
1. <u>Roof ponding</u>					
Ponding depth (a) 0.5 in	71	125.4	0.2	265,200	5.7
(b) 1.0 in	142	103.4	17.7	238,400	15.2
(c) 1.5 in	213	87.0	30.8	210,400	25.2
(d) 2.0 in	260	87.0	30.8	206,400	26.6
(e) 2.0 in*	260	79.1	37.1	186,400	33.7
2. <u>Site storage of roof runoff</u>					
Outflow rate (a) 0.08 cfs	24	117.6	6.3	280,800	0.1
(b) 0.04 cfs	63	102.3	17.8	269,800	4.0
(c) 0.02 cfs	109	94.6	24.7	240,400	14.5
(d) 0.01 cfs	158	90.8	27.8	223,400	20.6
(e) 0.01 cfs*	158	82.9	34.1	203,400	27.6
3. <u>Catch basin storage</u>					
Outflow rate (a) 0.40 cfs	520	115.8	7.9	277,200	1.4
(b) 0.30 cfs	800	108.7	13.5	265,200	5.7
(c) 0.20 cfs	1,100	99.3	21.0	240,100	14.6
(d) 0.10 cfs	1,500	87.9	30.1	203,400	27.6
(e) 0.10 cfs*	1,500	79.2	37.0	183,400	34.8
4. <u>Holding reservoir at outfall (constant outflow)</u>					
Outflow rate (a) 90.0 cfs	25,000	90.0	28.5	281,200	0.0
(b) 60.0 cfs	71,700	60.0	52.2	242,300	13.8
(c) 30.0 cfs	152,300	30.0	76.2	152,300	45.9

Continued.....

Table 11 (Cont'd)

Effects of Storage on Combined Sewer Flows from Subdivision

Storage Option	Unit Storage Volume (cu ft)	Peak Outflow (cfs)	Reduction in Peak Outflow(%)	Outflow Volume** (cu ft)	Reduction in Outflow Volume(%)
5. <u>Holding reservoir at outfall (orifice outflow control)</u>					
Reservoir area = 10,000 sq ft					
Orifice diameter (a) 18 in	176,000	36.1	71.2	111,900	60.2
(b) 30 in	100,300	72.4	42.6	222,300	21.0
Reservoir area = 20,000 sq ft					
Orifice diameter (a) 18 in	203,000	27.0	78.5	79,900	71.5
(b) 30 in	136,000	57.5	54.2	172,000	38.8
6. <u>Roof ponding and catch basin storage</u>					
Ponding depth = 2 in	260	65.2	48.1	190,400	32.3
Catch basin outflow rate = 0.30 cfs	800				

* Industrial area runoff reduced by 40%

** Outflow volumes for 80 minute period.

slightly smaller than for storm sewers. For the same reason, larger holding reservoir storages are needed to reduce combined flows to the same level as stormwater flows.

Although the results show that small storage volumes are not very effective in reducing peak flows for the design storm, small amounts of storage will reduce pollution from combined sewer overflows for storms of lesser magnitude. For example, small depths of roof ponding will retard runoff from low intensity storms and a greater proportion of total runoff volume will be diverted to interceptors for treatment.

5.4 Storage versus Peak Flow for Holding Reservoirs

In view of the obvious advantages of holding reservoirs in terms of large-scale flow modification, it seems worthwhile to pursue further the properties of this type of storage. Figure 27 shows the relationship between required storage volume and peak outflow from the subdivision for a holding reservoir located at the separate storm sewer outfall. Curves are drawn for holding reservoirs both with constant outflow and with outflow controlled by a 30 inch diameter orifice.

For any level of flow reduction, the storage volume required in a holding reservoir with orifice outflow control is greater than that in a reservoir with constant outflow. Furthermore, the outflow is governed by the dimensions of the storage, according to the orifice formula. The relationship between flow and depth of storage is shown in Figure 27 for a 30 inch diameter orifice.

If, for example, it is desired to reduce peak storm sewer flow from the subdivision to 50 cfs, 85,000 cu ft of storage will be required in a reservoir with pumped outflow. A reservoir with gravity outflow controlled by a 30 inch orifice must have 142,000 cu ft of storage. Furthermore, the depth of storage must not exceed 5.2 feet or the outflow will be more than 50 cfs. Then the reservoir area must be greater than $142,000/5.2 = 27,400$ sq ft, i.e. more than half an acre.

A family of curves such as in Figure 27 may be drawn for orifices of different diameters. Then the orifice size may be selected to give maximum possible flow reduction within the constraints of available area and available elevation. In choosing between the gravity and pumped systems, the costs of land and of storage construction must be traded

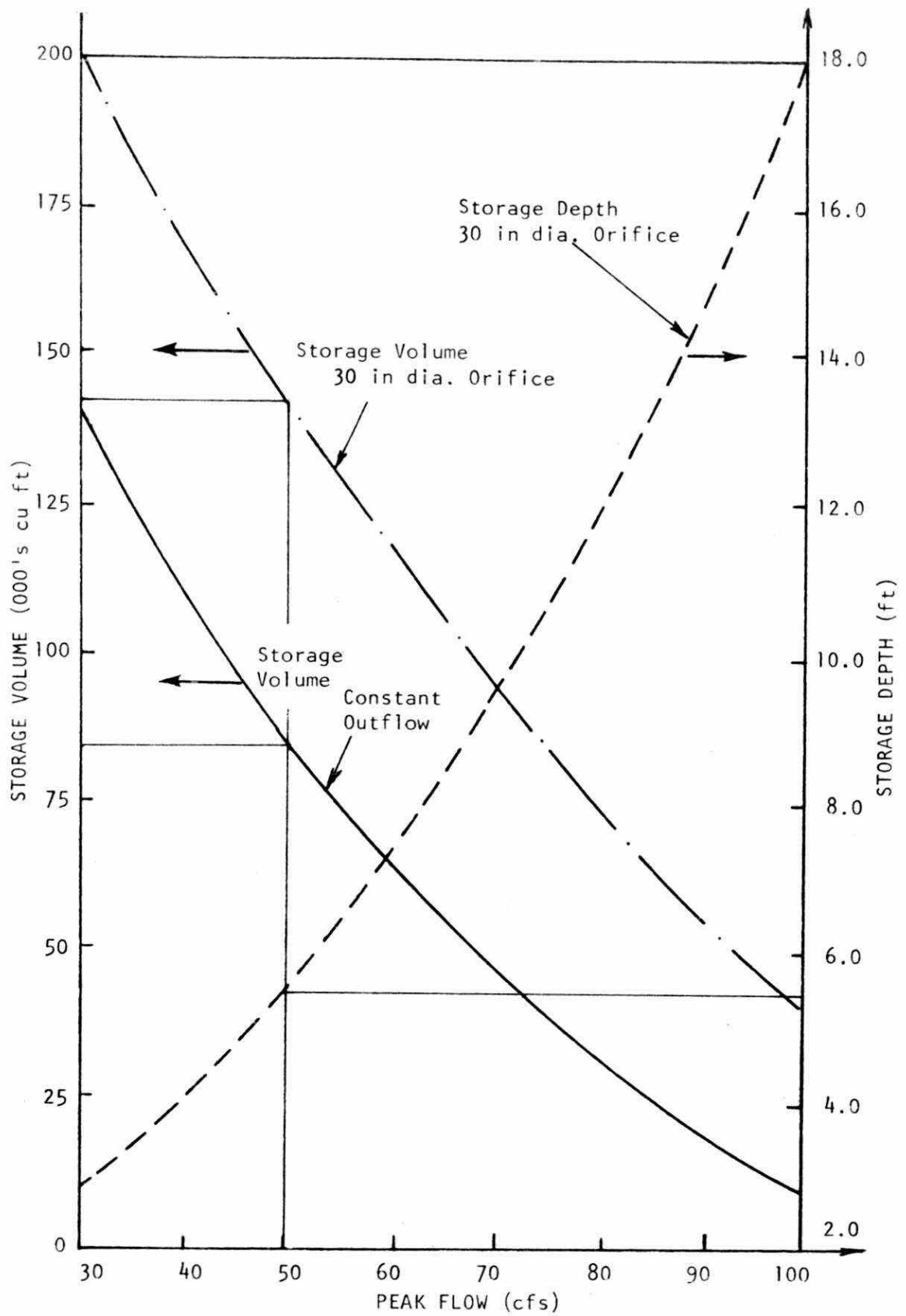


FIGURE 27. STORAGE VOLUME AND DEPTH vs PEAK FLOW FOR STORMWATER HOLDING RESERVOIR

off against the capital and operating costs of pumping, having regard also to reliability.



6. COSTS OF SEWER SYSTEM COMPONENTS

6.1 Separate System Costs

The principal components of conventional sewer systems are sewer pipes, manholes and catch basins. When storage of the types investigated in this study is incorporated, storage elements become important components in the system. The costs of these components are examined in the following sections.

6.1.1 Sewer pipe costs (separate)

The costs of excavation, pipe supply and pipe installation form the major costs in conventional gravity sewer systems. A U.S. study assembled cost estimates for five contracts with pipe diameters ranging from 8 inches to 24 inches placed in different types of soils at depths of up to 20 feet (25). Multiple regression equations were fitted to the data and equations found to give good fit to all data groups were of the form:

$$C_{\text{pipe}} = x + yD^2 + zH^2$$

where:

C_{pipe} is unit cost of excavation, pipe supply
and pipe installation (\$/ft);

D is internal diameter of pipe (ft);

H is average depth to pipe invert (ft); and,

x, y, z are constants.

A similar equation was used to compare the estimated unit costs with tender prices for the original separate sewer system of the subdivision studied in this research. Because the original system incorporated common trench construction, i.e. both the sanitary sewer and the storm sewer at the same depth in a single trench, two terms were introduced for the diameters of the two pipes. An equation found to give good agreement with the tender prices is:

$$C_{\text{sep}} = 4.8 + 4.0 D_{\text{san}}^2 + 2.3 D_{\text{stm}}^2 + .041H^2$$

where: C_{sep} is unit cost of separate sewers (i.e. sanitary plus storm) (\$/ft);
 D_{san} is diameter of sanitary sewer (ft);
 D_{stm} is diameter of storm sewer (ft); and,
 H is average depth to pipe invert (ft).

The costs are based on 1973 price levels.

The estimated costs and tender prices of the original sewers are compared in Table 12 for the main sewer line. The 18 sewers had been aggregated to 11 items for tendering purposes. The cost estimating equation gives results which are within ten per cent of the tender prices for all items, and for the whole sewer network, the results are within ten per cent for 41 of 44 items. Using this equation, the total sewer pipe cost for the subdivision is \$328,700 compared with a tender price of \$336,200, so that the estimating equation gives good agreement.

Table 12
Comparison of Cost Estimates and Tender Prices
for Subdivision Separate Sewer Pipes

Sewer No.	Sanitary Sewer Diameter (in)	Storm Sewer Diameter (in)	Average Depth (ft)	Tender Price (\$/ft)	Cost Estimate (\$/ft)	Error in Cost Estimate (%)
1	15	15	13.1	20.58	21.68	5.3
2	15	18	12.5	22.15	22.63	2.2
3,4	15	21	12.4	22.94	24.40	6.4
5	15	27	15.9	32.98	33.06	0.2
6	15	30	17.4	39.38	37.84	-3.9
7,8	15	33	17.3	41.53	40.71	-2.0
9,10,11	15	36	15.6	43.98	41.73	-5.1
12	15	42	14.8	50.87	48.21	-5.2
13	15	42	16.4	50.98	50.25	-1.4
14,15,16	18	48	20.5	69.78	67.83	-2.8
17,18	18	54	18.6	80.30	74.56	-7.1

The cost estimating equation was applied to evaluate the sewer costs for some of the storage options in Chapter 5. Because the storm sewer flows predicted by the subdivision hydrograph model are greater than those using the rational method, the required pipe diameters are larger than those in the original design for some storm sewers. As the level of local storage is increased, peak stormwater flows are reduced and reductions in storm sewer sizes are possible. Table 13 shows the required storm sewer diameters in the main sewer line for:

no storage

storage option 1(d): no roof runoff

storage option 6 : no roof runoff and catch basin outflow
rate = 0.30 cfs

storage options 7,8: holding reservoir at upstream end of
sewer no. 14 with outflow rate = 50 cfs.

Reductions in pipe diameters by at least one commercial size are achieved when roof runoff is eliminated, with further reductions for additional catch basin storage. When a holding storage is incorporated above sewer no. 14, the required sizes of sewers no. 14 to 17 are reduced. No reduction in subdivision sewer sizes is possible with a holding reservoir at the subdivision outfall.

The sewer slopes were assumed to be unchanged for each storage option. As pipe sizes are reduced for a given slope, full-flow velocities fall. A ten inch pipe for storm sewer no. 1 would have a full-flow velocity of 2.2 ft/sec, which is below the adopted minimum value of 3 ft/sec for storm sewers. This result would occur for several of the smaller lateral sewers but since no sewers had full-flow velocities less than 2 ft/sec, the original slopes were not changed. In cases where full-flow velocities in main sewers are close to 3 ft/sec, reductions in sewer sizes by means of storage might require that the slopes of these sewers be increased in order to maintain acceptable velocities. Then the cost savings with smaller pipes would be offset by higher excavation costs.

The total sewer pipe cost estimates for each of the storage options are shown in Table 14.

Table 13

Storm Sewer Diameters for Various Storage Options

Storm Sewer No.	Storm Sewer Diameters (in)			
	No Storage	Storage Option 1(d)	Storage Option 6	Storage Options 7,8
1	15	10	10	15
2	18	12	12	18
3	24	18	15	24
4	24	21	18	24
5	30	24	21	30
6	30	24	21	30
7	33	27	24	33
8	33	27	24	33
9	36	30	24	36
10	36	30	24	36
11	39	30	27	39
12	42	36	33	42
13	48	39	33	48
14	54	42	39	48
15	54	48	39	48
16	54	48	39	48
17	54	48	39	48
18	54	48	42	54

Table 14

Separate Sewer Pipe Costs for Various Storage Options

	Separate Sewer Pipe Cost (\$)	Reduction (%)
No Storage	344,000	----
Storage option 1(d)	300,000	12.8
Storage option 6	272,700	20.7
Storage options 7,8	333,800	3.0

Roof ponding alone would provide only a 12.8 per cent saving in sewer pipe costs for the subdivision. Similar savings could be expected for site storage of roof runoff or catch basin storage. Both roof storage and catch basin storage would be needed to provide greater savings. Although storm sewers no. 14 to 17 have large diameters and high unit costs, their combined length is only a small fraction of the total length of storm sewers within the subdivision. Hence a size reduction in these sewers has only a small effect on total costs.

6.1.2 Manhole and catch basin costs (separate)

As well as sewer pipes, manholes and catch basins also contribute significantly to total costs of sewer systems. The relative costs of the three components in the original separate sewer system for the subdivision were:

	<u>Costs (\$)</u>	<u>(%)</u>
sewer pipes	336,200	63.8
manholes	133,800	25.4
<u>catch basins</u>	<u>56,800</u>	<u>10.8</u>
Total	526,800	100.0

The combined cost of manholes and catch basins is 36.2 per cent of the total cost of the original system and is unaffected by incorporation of

storage, except where catch basin storage is involved. A cost comparison for the different storage options, using total costs of sewer pipes, manholes and catch basins, is shown in Table 15.

Table 15

Total Separate Sewer System Costs for Various Storage Options

	Total Costs (\$)	Reduction (%)
No Storage	534,600	----
Storage option 1(d)	490,600	8.2
Storage option 6	463,300	13.4
Storage options 7,8	524,400	1.9

The results indicate that only relatively small savings in subdivision sewer costs are achieved by local storage. For a regional study, potential cost savings in downstream trunk sewers should also be considered. If stormwater flows are diverted to a water pollution control plant, possible savings in treatment capacities and reductions in pollution loads on receiving waters may be the most important benefits of storage.

6.1.3 Costs of storage (separate)

The costs of storage must also be included in a total cost comparison. A thorough search of the literature failed to provide information on which even a rough estimate of storage costs could be based. However, it is possible to make some inferences concerning the economic feasibility of local storage by examination of the costs of the other system components: sewer pipes, manholes and catch basins.

Table 16 shows that roof ponding (storage option 1(d)) provides a saving of \$44,000 in subdivision sewer costs. If downstream cost savings are ignored, it may be stated that for roof ponding to be worthwhile, the extra costs involved must be less than \$44,000. Since there are 383 houses in the subdivision, the extra cost per house must be less than \$115. A similar figure would apply for site storage of roof runoff.

If storage was incorporated at each of the 129 catch basins to provide the same total cost reduction, the extra cost per catch basin must be less than \$341. If catch basin storage is provided in addition to roof ponding, a further reduction in total sewer costs of \$27,300 is achieved. Then the extra cost per catch basin must be less than \$211.

Table 16

Burial Costs and Erection Costs
vs. Storage Capacity for Holding Reservoirs

Storage Capacity (U.S. gallons)	Burial Costs (\$/U.S. gallon)	Erection Costs (\$)
100,000	.0480	13,000
500,000	.0390	30,000
1,000,000	.0310	53,000
2,000,000	.0230	110,000

Some data on costs of holding reservoirs has been presented in the literature. The EPA model calculates total reservoir storage costs as the sum of capital costs, annual maintenance costs and storm event costs, e.g. pumping costs for a pumped outflow and costs of chemicals if water quality improvement is desired (16). Capital costs are taken as the sum of land costs and construction costs (excavation, lining, cover, etc.). Recommended default values are \$20,000/acre for land costs and \$3.00/cu yd for construction costs. The latter figure is appropriate only for very large reservoirs with capacities of several million gallons. For smaller reservoirs, economies of scale may be expected to be significant; i.e. unit construction costs decrease as the size of the reservoir increases.

A more detailed breakdown of capital costs of holding reservoirs has been presented in a study of the beneficial use of storm-water (9). Storage costs were considered to be made up of the following:

1. Land acquisition costs;

2. Site preparation costs (clearing and grubbing, rough grading, etc.);
3. Burial costs (excavation and backfill for tanks installed below grade); and,
4. Erection costs (foundations, tank erection, painting of metal parts, etc.).

Burial costs and erection costs vary with storage capacity and typical values are listed in Table 16, with costs based on 1967-68 price levels. The erection costs are estimated costs for steel tanks constructed above grade. For tanks below grade, 15 per cent should be added to the erection costs.

The costs in Table 16 were used to provide a preliminary estimate of the costs of holding reservoirs at the subdivision outfall, i.e. storage options 4 and 5 (see Table 9). Burial and erection costs for the appropriate storage capacities were estimated by interpolation between values listed in Table 16. Land acquisition costs and site preparation costs were assumed to be \$20,000/acre and \$15,000/acre, respectively. For storage option 4 with a constant outflow from the reservoir, the dimensions of the storage are not constrained. Reservoir areas of 5,000 sq ft, 10,000 sq ft and 15,000 sq ft were assumed for cases (a), (b), and (c), respectively. For the purpose of calculating required land area, the storage was assumed to be in the form of a cylindrical tank with a 15-foot border around the tank. The cost estimates are summarized in Table 17.

At the assumed unit cost levels, land acquisition costs and site preparation costs are relatively small compared to burial and excavation costs. It must be emphasized that the estimates merely provide a guide to the order of costs for holding reservoirs. The actual costs for a given location will depend largely on factors specific to that location. For example, land costs may make reservoirs economically unattractive in high land value areas.

Table 17Cost Estimates for Holding Reservoirs at Subdivision Outfall

Storage Option	Reservoir Area (sq ft)	Storage Volume (cu ft)	Costs (thousands of dollars)					Cost/Unit Volume (\$/cu ft)
			Land Acquisition	Site Preparation	Burial	Erection	Total	
4(a)	5,000	20,500	3.3	2.4	7.2	17.6	30.5	1.48
4(b)	10,000	64,600	5.9	4.4	19.0	33.6	62.9	0.97
4(c)	15,000	140,400	8.5	6.4	32.3	64.7	111.9	0.80
5(a)	10,000	164,300	5.9	4.4	35.9	76.3	122.5	0.73
5(b)	10,000	95,200	5.9	4.4	25.4	45.5	81.2	0.85
5(c)	20,000	191,200	11.0	8.3	39.4	89.2	147.9	0.77
5(d)	20,000	129,000	11.0	8.3	30.5	59.2	109.0	0.85

The unit cost of storage is a maximum of \$1.48/cu ft for storage option 4(a) and a minimum of \$0.73/cu ft for option 5(a). Although option 5(c) has a greater storage requirement than option 5(a), the unit cost is slightly higher because of a greater land area. The results indicate that there are significant economies of scale for storage so that the costs of small units such as underground tanks on residential lots might be excessive relative to the benefits from peak flow reduction.

Holding reservoirs within the subdivision would not be economically attractive and, in fact, the 100-acre subdivision may be too small an area for a reservoir. For low density residential areas with relatively low imperviousness, the economically efficient drainage area for a reservoir may be of the order of 1,000 acres or more. The general problem is to find the optimum locations and sizes of holding reservoirs in the sewer network.

6.2 Combined System Costs

The following sections describe the estimation of costs of components of an alternative combined sewer system for the subdivision. Details of the combined sewers were presented in Chapter 4.

6.2.1 Sewer pipe costs (combined)

For evaluating costs of combined sewer pipes, an estimating equation of the same form as that for separate sewers was developed. In the case of a combined sewer, there is only one pipe in a trench and only one term is required to represent pipe diameter. For calibration of the estimating equation, typical cost data were obtained from a firm of consulting engineers (26). The following equation was found to give good fit to the data:

$$C_{com} = 2.8 + 2.0 D_{com}^2 + .045 H^2$$

where: C_{com} is unit cost of excavation, pipe supply and pipe installation (\$/ft);
 D_{com} is diameter of combined sewer (ft); and,
 H is average depth to pipe invert (ft).

The costs are based on 1973 price levels.

As a check on accuracy, the equation was applied to estimate the costs of the original system, but with sanitary sewers and storm sewers in separate trenches. The following results were obtained:

	<u>Costs (\$)</u>
sanitary sewer pipes	155,500
<u>storm sewer pipes</u>	<u>252,100</u>
Total	407,600

The estimated cost of a separate system with separate trench construction is \$71,400 greater (or 21.3 per cent) than the actual cost of the separate system with common trench construction. This result accurately reflects the cost savings using current techniques of common trench construction.

The total costs of combined sewer pipes for the subdivision were evaluated for no storage and for storage options 1(d) and 6, and the results are summarized in Table 18.

Table 18

Combined Sewer Pipe Costs for Various Storage Options

	Combined Sewer Pipe Cost (\$)	Reduction (%)
No Storage	331,500	---
Storage Option 1(d)	294,500	11.2
Storage Option 6	277,200	16.4

The estimated cost of combined sewer pipes for the case of no storage is \$12,500 less (or 3.6 per cent) than the estimated cost of separate sewer pipes. The per cent cost savings resulting from storage are slightly less than the corresponding savings with separate sewers.

6.2.2 Manhole and catch basin costs (combined)

The costs of catch basins in a combined sewer system are the same as the costs in a separate system. Because only one structure is required at each manhole location, the costs of manholes will be somewhat less than the costs in a separate system. But since combined sewers are deeper than separate sewers, the depths of manholes are increased and some of the cost savings are offset. The cost of manholes in the combined system is assumed to be 25 per cent less than that in the separate system. Estimated component costs in the combined sewer system for the case of no storage are as follows:

	<u>Costs (\$)</u>	<u>(%)</u>
sewer pipes	331,500	67.9
manholes	100,300	20.5
<u>catch basins</u>	<u>56,800</u>	<u>11.6</u>
Total	488,600	100.0

The effect of storage on total combined sewer costs is shown in Table 19. The estimated total cost without storage is \$46,000 less (or 8.6 per cent) than the estimated cost of a separate system. As for the separate system, the cost savings in subdivision sewers resulting from storage are relatively small.

Table 19

Total Combined Sewer System Costs for Various Storage Options

	<u>Total Cost</u> <u>(\$)</u>	<u>Reduction</u> <u>(%)</u>
No Storage	488,600	---
Storage Option 1(d)	451,600	7.6
Storage Option 6	434,300	11.2

It should be noted that possible downstream costs associated with the combined sewers have not been considered. The combined sewer depth at the subdivision outfall is about six feet greater than the separate sewer depth. If it is intended that the flows from the subdivision enter an existing trunk sewer, the feasibility of combined sewers may depend on the invert level of the trunk sewer. If the combined sewer lies below the trunk sewer, pumping will be required and the potential cost advantages of combined sewers will almost certainly be lost.

6.2.3 Costs of storage (combined)

The costs of storage for a combined sewer system would be similar to those for a separate system and the same comments apply. In the case of combined sewers, downstream benefits may be most important. Combined sewer overflows are a major source of pollution of receiving waters. Incorporation of storage in new residential subdivisions would reduce peak flows from these areas during storms and would enable a greater proportion of the combined sewer flows to be treated. Hence, detrimental impacts on receiving waters would be lessened.

7. COMPARISON OF STORAGE METHODS

7.1 Technical Feasibility

In Chapter 5, it was shown that for a two-year design storm, provision of roof ponding in the residential area would reduce peak storm sewer flows from the subdivision by 31.5 per cent. If additional roof ponding were provided in the industrial areas, peak flows would be reduced by 37.9 per cent. Similar reductions would be achieved by site storage of roof runoff with outflow rate limited to 0.01 cfs, and by catch basin storage with outflow rate limited to 0.10 cfs. The storage requirements would be 158 cu ft for site storage and up to 1,500 cu ft for catch basins. Outflow from a holding reservoir at the subdivision outfall could be made as small as desired, but with corresponding increases in storage requirements. Similar flow reductions could be expected for a combined sewer system as for the existing separate sewer system.

This research has been concerned primarily with evaluation of the storage methods from the viewpoint of peak flow reductions. The above results indicate that local storage would almost certainly be worthwhile from this viewpoint. But the practicability of storage will depend also on construction and operational feasibility and cost.

7.1.1 Roof ponding

Roof ponding would require the use of flat roof construction, which would be a departure from the normal practice of gabled roof construction for low-rise residential buildings. There should be few structural problems with flat roofs. The maximum depth of ponding would be limited by the design load, usually based on live load or snow load. It may be possible to store all the rainfall from a two-year storm but not, for example, from a 100-year storm. Overflow from roof storage would have to be controlled so that it would enter the drainage system without causing damage.

It would be necessary to ensure the availability of roof storage at the beginning of a storm. In general, it might not be possible to rely on evaporation to remove ponded water and some means of removal after each storm would be required. Roof drainage design could

provide for gradual depletion of ponded water. Since the storage will generally be located on privately-owned buildings, the question of responsibility for continuous operating effectiveness will need to be considered.

7.1.2 Site storage of roof runoff

A site storage facility such as an underground tank could control the rate at which roof runoff enters the sewer. Since storm or combined sewers are generally more than ten feet below ground level, there is sufficient depth for a tank within a gravity drainage system. Orifices or weirs for controlling flow rates would have to be protected against clogging by means of grates, for example. Accessibility to the storage tank for periodic cleaning would be an important consideration.

7.1.3 Catch basin storage

For effective peak flow reduction, storage requirements at catch basins with high inflows would be considerably greater than the volumes provided in standard-sized catch basins. Although the storage requirements are different for each of the catch basins, some economy might be achieved by incorporating three or four different-sized catch basins in the subdivision.

The three storage methods considered above will modify the quality characteristics of stormwater flows into sewers. Temporary detention of water will cause deposition of sediment and transport of some pollutants will be reduced. Where no downstream treatment is provided, e.g. for storm sewer flows or combined sewer overflows, source control of pollutants may be a major benefit of storage. However, the costs associated with source control, e.g. costs of catch basin cleaning, must also be considered.

7.1.4 Holding reservoirs

The technical feasibility of holding reservoirs as a means of peak flow reduction has been well established. The form of the reservoir might be a constructed facility, such as an underground tank, or, if the

terrain is suitable, a natural retention basin. Because of water quality problems, a natural basin would not be feasible for a combined sewer system.

The performance of the sewer system incorporating storage should be assessed for more intense storms than the design storm, e.g. the 100-year storm. Both the sewers and the storages will be surcharged and the system should be designed to minimize undesirable flooding.

7.2 Economic Attractiveness

In Chapter 6, the cost of a combined sewer system for the subdivision was estimated to be \$488,600 compared with \$534,600 for a separate system, i.e. a reduction of 8.6 per cent. Possible downstream costs resulting from the extra depths of combined sewers were not considered.

The combined sewer design was based on the same two-year storm as the separate storm sewer design. But because of the raw sewage content, surcharge would be less acceptable for combined sewers than for storm sewers. In practice, combined sewers should be sized to accommodate a more intense storm, e.g. a five-year storm. Under these circumstances, combined sewer costs would be greater than the above estimate.

Roof ponding was estimated to provide a saving of \$44,000 for the separate sewer system. Ignoring possible downstream benefits, the extra cost of flat roof construction would need to be less than \$115 per roof for ponding to be economical. A similar unit cost would apply for site storage of roof runoff. For catch basin storage, the extra cost per catch basin must be less than \$341. The unit costs of stormwater holding reservoir storage in the form of underground steel tanks were estimated to decrease from \$1.48/cu ft for a 20,500 cu ft reservoir to \$0.73/cu ft for a 164,300 cu ft reservoir.

Since the cost savings with combined sewers are quite small, it would appear that their use in new residential subdivisions is not justified. The problems of adequate movement of dry weather flows would outweigh the cost savings. Furthermore, it may prove more economical to effect quality improvement of stormwater by means of upstream storage than by transport of combined sanitary and stormwater flows to downstream

treatment works. However, these conclusions follow from the study of a single subdivision and may not be generally applicable.

The lack of cost data makes a direct cost comparison of the different storage methods difficult. The initial cost of a flat roof may be less than that of a gabled roof. High unit costs for small storage installations may make underground site storage of roof runoff and catch basin storage uneconomical.

If constructed facilities are not provided, other methods of runoff control might be possible. Instead of underground storage, part of the roof runoff might be diverted on to lawns. A recent survey of Canadian storm sewer design practice found that roof drainage onto property was allowed by 21 of 37 municipalities surveyed (27). Instead of catch basin storage, runoff might be temporarily ponded on streets by restricting capacities of catch basin inlets. These methods would eliminate storage construction costs but inconvenience would result during heavy storms.

Holding reservoirs appear to be the most practical method of storage of stormwater. Land acquisition costs may be a limiting factor in high land value areas. For new residential subdivisions, the required land would need to be reserved at an early stage of development. Nine storage ponds have been incorporated in residential subdivisions in Winnipeg (27).

The major benefits of a holding reservoir, in terms of reduced trunk sewer sizes and lower pollution loads on receiving waters, accrue downstream from the reservoir. Since this study examined only a small subdivision, estimation of these benefits was not possible. An indication of the possible savings in sewer costs is provided by a 1500 acre development in Earth City, Missouri (11). A drainage system incorporating stormwater storage in the form of lakes was estimated to cost two million dollars, exclusive of land costs, whereas a system without storage would have cost five million dollars.

7.3 Future Work

The results of this research indicate that local storage in new residential subdivisions may be effective. However, operational reliability needs to be established. Field testing of different local storage methods should be carried out. Incorporation of storage facilities in a small test area of a subdivision and subsequent performance monitoring would not require a large commitment of resources and should enable some firm conclusions to be drawn.

Present industrial and commercial developments have relatively high imperviousness. A single roof may cover an area of several thousand square feet and large paved parking lots are common. Because of high runoff rates, local storage may be quite effective in industrial areas and the possibilities for implementation of storage in these areas should be investigated.

If present urbanization patterns continue, the provision of facilities for wastewater collection, transport and treatment, and their integration with existing facilities will pose major problems. Future work should be oriented towards systems studies for large areas to determine the most effective means of stormwater management on a regional basis.

A high priority should be given to implementation of storage methods such as holding reservoirs, for which engineering design practices are well established. By requiring developers to limit runoff rates and to maintain the quality of runoff from new subdivisions, the use of stormwater storage would be encouraged and the operating effectiveness of storage could be evaluated on a continuing basis.

ACKNOWLEDGEMENTS

This research was supported under Department of Supply and Services Contract No. OSS3-0258 for the Environmental Protection Service, Department of the Environment.

The design and cost data for the residential subdivision which was studied in the research were provided by R.V. Anderson Associates Limited, Consulting Engineers and Planners, whose cooperation is gratefully acknowledged.

Dr. P. Wisner, of James F. MacLaren Limited, Environmental Engineers and Scientists, provided helpful suggestions during the course of the research.

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APPENDIX I

ABBREVIATIONS

APPENDIX I

ABBREVIATIONS

cu ft	cubic feet
cu yd	cubic yards
cfs	cubic feet per second
ft	feet
ft/sec	feet per second
in	inches
in/hr	inches per hour
min	minutes
sec	seconds
sq ft	square feet

APPENDIX II

PROGRAM LISTING FOR SUBDIVISION HYDROGRAPH MODEL

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C      SUBDIVISION HYDROGRAPH MODEL
C      A PROGRAM FOR CALCULATING RUNOFF FROM RAINFALL AND FLOWS IN
C      EITHER STORM OR COMBINED SEWERS IN RESIDENTIAL SUBDIVISIONS
C      A STEPWISE PROCEDURE IS USED IN WHICH RUNOFF AND SEWER FLOWS
C      ARE CALCULATED FOR SUCCESSIVE TIME INTERVALS

      DIMENSION TYPE(5),S(5),RUF(5),DD(5),TL(5),CON(5)
      DIMENSION NUMCB(129),WROAD(129),WDRIV(129),ARLOT(129),DGUT(129),
1SGUT(129),IR(129),QLOTS(129,161),QGOUT(129,161),Q(5,161),TLOT(129)
      DIMENSION NUMSL(56),NUMUPM(56),NUMDOM(56),SEWL(56),SEWV(56),
1NRI(56),QSAN(56),NROOFS(56),NCBS(56),NUMCS(5,56),DCS(5,56),
2QUPM(56,161),QROOF(56,161),QDOM(56,161),ROOF(56),QSEW(56,161)
      DIMENSION RAIN(161),RINF(161),TCL(161),TIME(161),FLOW(161),
1TIMFLS(56),MHE(4),QME(4),TME(4),QSINRF(161),LOC(10),QSTO(10)

C      READ IN DATA
C      TIMES: STORM DURN, TIME TO PEAK, TIME INTERVAL, ROOF-SEWER LAG
C      RAINFALL: COEFFTS FOR AVE INTENSITY-DURATION FORMULA
C      INFILTRATION: MAX RATE, MIN RATE, DECAY RATE
C      SUBCATCHMENTS: TYPE, SLOPE, ROUGHNESS, DEPRESSION DEPTH, LENGTH
C      GUTTERS: ROUGHNESS, SIDE SLOPE
C      ROOFS: ROOF AREA/LOT
C      CATCH BASINS: ROAD AND DRIVEWAY WIDTHS, PERVIOUS AREA SIZE AND
C      LENGTH, GUTTER LENGTH AND SLOPE
C      SEWERS: UPSTREAM AND DOWNSTREAM MANHOLES, LENGTH, FULL-FLOW VEL,
C      NO OF ROOFS, PEAK SANITARY FLOW
C      CATCH BASIN-SEWER CONNECTIVITY
C      EXTERNAL FLOWS: MANHOLE, PEAK EXT FLOW, TIME TO PEAK
C      ANTECEDENT CONDITIONS: INFILTRATION, PONDING

      READ (5,100) TIM,TBP,TINT,TRLAG
100  FORMAT (4F10.0)
      WRITE (6,110) TIM,TBP,TINT,TRLAG
110  FORMAT (7X,'TIM',7X,'TBP',6X,'TINT',5X,'TRLAG',/,4F10.2)
      READ (5,100) A,B,C
      WRITE (6,115) A,B,C
115  FORMAT (9X,'A',9X,'B',9X,'C',/,F10.0,F10.1,F10.0)
      READ (5,100) FOIN,FCIN,DKIN
      WRITE (6,120) FOIN,FCIN,DRIN
120  FORMAT (6X,'FOIN',6X,'FCIN',6X,'DRIN',/,3F10.2)
      WRITE (6,125)
125  FORMAT (6X,'TYPE',9X,'S',7X,'RUF',8X,'DD',8X,'TL')
      DO 130 I=1,5
      READ (5,135) TYPE(I),S(I),RUF(I),DD(I),TL(I)
135  FORMAT (A5,4F10.0)
      WRITE (6,140) TYPE(I),S(I),RUF(I),DD(I),TL(I)
140  FORMAT (5X,A5,3F10.4,F10.1)
130  CONTINUE
      READ (5,100) RUFG,SSG
      WRITE (6,150) RUFG,SSG
150  FORMAT (6X,'RUFG',7X,'SSG',/,2F10.4)
      READ (5,100) URFA
      WRITE (6,155) URFA
155  FORMAT (11X,'URFA',/,5X,F10.0)
      READ (5,170) NCB,NSL
170  FORMAT (2I5)
      WRITE (6,200) NCB,NSL
200  FORMAT (2X,'NCB',2X,'NSL',/,2I5)
      WRITE (6,210)
210  FORMAT (5X,'NUMCB',3X,'IR',5X,'WROAD',5X,'WDRIV',5X,'ARLOT',6X,
1'TLOT',6X,'DGUT',6X,'SGUT')

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```

      IF (NCB.EQ.0) GO TO 245
      DO 220 J=1,NCB
      READ (5,230) NUMCB(J),IR(J),WROAD(J),WDRIV(J),ARLOT(J),TLOT(J),
10GUT(J),SGUT(J)
230  FORMAT (2I5,6F8.0)
      WRITE (6,240) NUMCB(J),IR(J),WROAD(J),WDRIV(J),ARLOT(J),TLOT(J),
10GUT(J),SGUT(J)
240  FORMAT (5X,2I5,5F10.0,F10.4)
220  CONTINUE
245  WRITE (6,250)
250  FORMAT (10X,'NUMSL',4X,'NUMUPM',4X,'NUMDOM',6X,'SEWL',6X,'SEWV',4X
1,'NRDOFS',7X,'NRI',6X,'QSAN')
      DO 260 L=1,NSL
      READ (5,270) NUMSL(L),NUMUPM(L),NUMDOM(L),SEWL(L),SEWV(L),
1NRDOFS(L),NRI(L),QSAN(L)
270  FORMAT (3I5,2F10.0,2I5,F10.0)
      WRITE (6,280) NUMSL(L),NUMUPM(L),NUMDOM(L),SEWL(L),SEWV(L),
1NRDOFS(L),NRI(L),QSAN(L)
280  FORMAT (5X,3I10,F10.0,F10.2,2I10,F10.3)
260  CONTINUE
      WRITE (6,290)
290  FORMAT(10X,'NUMSL',4X,'NUMCS1',6X,'DCS1',4X,'NUMCS2',6X,'DCS2',
14X,'NUMCS3',6X,'DCS3',4X,'NUMCS4',6X,'DCS4',4X,'NUMCS5',6X,'DCS5')
      DO 300 L=1,NSL
      READ (5,310) NCBS(L),(NUMCS(M,L),DCS(M,L),M=1,5)
310  FORMAT (5X,I5,5(I5,F6.0))
      NM=NCBS(L)
      IF (NM.EQ.0) GO TO 321
      WRITE (6,320) NUMSL(L),(NUMCS(M,L),DCS(M,L),M=1,NM)
320  FORMAT (5X,I10,5(I10,F10.0))
      GO TO 300
321  WRITE (6,320) NUMSL(L)
300  CONTINUE
      READ (5,170) NEF
      WRITE (6,330) NEF
330  FORMAT (7X,'NEF',/,I10)
      IF (NEF.EQ.0) GO TO 339
      WRITE (6,332)
332  FORMAT (7X,'MHE',7X,'QME',7X,'TME')
      DO 334 NE=1,NEF
      READ (5,336) MHE(NE),QME(NE),TME(NE)
336  FORMAT (I5,2F10.0)
      WRITE (6,338) MHE(NE),QME(NE),TME(NE)
338  FORMAT (5X,I5,2F10.2)
334  CONTINUE
339  READ (5,100) TANT,DEPIM,DEPPE
      WRITE (6,331) TANT,DEPIM,DEPPE
331  FORMAT (6X,'TANT',5X,'DEPIM',5X,'DEPPE',/,F10.2,2F10.4)

```

C
C

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      CALCULATE RAIN INTENSITY-TIME PATTERN FOR DESIGN STORM
      NINT=TIM/TINT
      DO 390 IT=1,NINT
      TCL(IT)=TINT*(FLOAT(IT)-0.5)
      IF (TCL(IT).GT.TBP) GO TO 391
      RAIN(IT)=A*((1.0-B)~((8./3.)*(TBP-TCL(IT)))*B+C)/
1      ((8./3.)*(TBP-TCL(IT)))*B+C)**2.0
      GO TO 390
391  RAIN(IT)=A*((1.0-B)*(8./5.)*(TCL(IT)-TBP))*B+C)/
1      ((8./5.)*(TCL(IT)-TBP))*B+C)**2.0
390  CONTINUE

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```

WRITE (6,392) TINT
392 FORMAT (/,10X,'RAINFALL(IN/HR) AT',F5.2,' MIN INTERVALS')
WRITE (6,394) (RAIN(IT),IT=1,NINT)
394 FORMAT (15(/,15X,12F8.3))

C
C      STEPWISE PROCEDURE FOR RUNOFF FROM ROADS, DRIVEWAYS
DO 399 I=1,5
399 CON(I)=1.486/RUF(I)*S(I)**0.5
DO 400 I=1,4
DEP1=DEPIM
DO 410 IT=1,NINT
DELDEP=RAIN(IT)/720.*TINT
DEP2=DEP1+DELDEP/2.0
IF (DEP2.LE.DD(I)) GO TO 420
Q(I,IT)=CON(I)*(DEP2-DD(I))*1.6667
GO TO 422
420 Q(I,IT)=0.0
422 DEP1=DEP1+DELDEP-Q(I,IT)*60.0*TINT/TL(I)
IF (DEP1.LT.0.0) DEP1=0.0
410 CONTINUE
400 CONTINUE

C
C      CALCULATE INFILTRATION RATE VS TIME
DO 430 IT=1,NINT
X=TCL(IT)+TANT
RINF(IT)=FCIN+(FOIN-FCIN)/EXP(X*DRIN)
430 CONTINUE

C
C      ITERATION THROUGH PROGRAM FOR EACH STORAGE DATA SET
READ (5,170) NDSETS
WRITE (6,630) NDSETS
630 FORMAT (/,4X,'NDSETS',/,I10)
DO 640 NDS=1,NDSETS

C
C      CALCULATE HYDROGRAPHS FOR EXTERNAL FLOW INPUTS
DO 340 L=1,NSL
DO 350 IT=1,NINT
350 QUPM(L,IT)=0.0
IF (NEF.EQ.0) GO TO 340
DO 360 NE=1,NEF
NINTS=TME(NE)/TINT
IF (NUMUPH(L).NE.MHE(NE)) GO TO 360
DO 370 IT=1,NINTS
370 QUPM(L,IT)=FLOAT(IT)/FLOAT(NINTS)*QME(NE)
NINTSA=NINTS+1
DO 380 IT=NINTSA,NINT
QUPM(L,IT)=(1.0-3.0/5.0*FLOAT(IT-NINTS)/FLOAT(NINTS))*QME(NE)
IF (QUPM(L,IT).LT.0.0) QUPM(L,IT)=0.0
380 CONTINUE
360 CONTINUE
340 CONTINUE

C
C      STEPWISE PROCEDURE FOR RUNOFF FROM PERVIOUS AREAS
DO 460 J=1,NCB
DEP1=DEPPE
DO 470 IT=1,NINT
RNET=RAIN(IT)-RINF(IT)
DELDEP=RNET/720.*TINT
DEP2=DEP1+DELDEP/2.0
IF (DEP2.LE.DD(5)) GO TO 480

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      QLOTS(J,IT)=CON(5)*((DEP2-DD(5))**1.6667*(DGUT(J)-WDRIV(J))
      GO TO 475
480  QLOTS(J,IT)=0.0
475  DEPI=DEPI+DELDEP-QLOTS(J,IT)*60.*TINT/ARLOT(J)
      IF (DEPI.LT.0.0) DEPI=0.0
470  CONTINUE
460  CONTINUE

C
C      STEPWISE PROCEDURE FOR GUTTER OUTFLOW HYDROGRAPH
      CG1=1.0+SQRT(1.0+1.0/SSG**2.0)
      DO 500 J=1,NCB
      CGUT=1.486/RUFG*SGUT(J)**0.5
      VOL1=0.0
      DO 510 IT=1,NINT
      IF (IR(J).EQ.1) GO TO 512
      QROAD=Q(1,IT)*WROAD(J)
      QDRIV=Q(3,IT)*WDRIV(J)
      GO TO 515
512  QROAD=Q(2,IT)*WROAD(J)
      QDRIV=Q(4,IT)*WDRIV(J)
515  QGIN=QROAD+QDRIV+QLOTS(J,IT)
      DELVOL=QGIN*60.*TINT
      VOL2=VOL1+DELVOL/2.0
      IF (VOL2.LE.0.0) GO TO 520
      GAREA=VOL2/DGUT(J)
      GDEPTH=SQRT(GAREA*2.0*SSG)
      GHYDR=GAREA/GDEPTH/CG1
      QGOUT(J,IT)=CGUT*GHYDR**0.6667*GAREA
      GO TO 525
520  QGOUT(J,IT)=0.0
525  VOL1=VOL1+DELVOL-QGOUT(J,IT)*60.*TINT
      IF (VOL1.LT.0.0) VOL1=0.0
510  CONTINUE
500  CONTINUE

C
C      CALCULATE ROOF RUNOFF HYDROGRAPH FOR SINGLE ROOF, SEWER
      DO 600 IT=1,NINT
      QSINRF(IT)=RAIN(IT)*URFA/43200.
      DO 610 L=1,NSL
610  QROOF(L,IT)=QSINRF(IT)*NROOFS(L)
600  CONTINUE

C
C      READ IN STORAGE OPTIONS AND LEVELS
C      0 NO STORAGE      1 ROOF PONDING      2 SITE STORAGE
C      3 CATCH BASIN STORAGE      4 HOLDING RESERVOIR
      READ (5,170) ISTORE
      WRITE(6,341) ISTORE
341  FORMAT (1H1,3X,'ISTORE',/,I10)
      IF (ISTORE.EQ.0) GO TO 342
      IF (ISTORE.EQ.1) GO TO 343
      IF (ISTORE.EQ.4) GO TO 344
      READ (5,100) QSTOUT
      WRITE (6,346) QSTOUT
346  FORMAT (4X,'QSTOUT',/,F10.3)
      GO TO 342
343  READ (5,100) DSTOR
      WRITE (6,345) DSTOR
345  FORMAT (5X,'DSTOR',/,F10.4)
      GO TO 342
344  READ (5,170) NSTORL

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        WRITE (6,347) NSTORL
347  FORMAT (4X,'NSTORL',/,I10)
        WRITE (6,348)
348  FORMAT (7X,'LOC',6X,'QSTO')
        DO 349 NST=1,NSTORL
        READ (5,336) LOC(NST),QSTO(NST)
        WRITE (6,338) LOC(NST),QSTO(NST)
349  CONTINUE
342  IF (ISTORE.EQ.0.OR.ISTORE.EQ.4) GO TO 690
C
C      ROUTE FLOWS THROUGH APPROPRIATE STORAGE
        CALL STORAG(ISTORE,DSTOR,QSTOUT,QSTO,NINT,TINT,LOC,
        1RAIN,QSINRF,QROOF,NCB,NUMCB,QGOUT,QUPM,NROOFS,NSL,NST,LMARK)
C
C      ROUTE FLOWS THROUGH SEWER NETWORK
690  WRITE (6,695)
695  FORMAT (//,5X,'NUMSL',5X,'SEWER FLOW',/)
        DO 700 L=1,NSL
        DO 700 IT=1,NINT
700  QDOM(L,IT)=0.0
        DO 710 L=1,NSL
        N1=NCBS(L)
        N2=N1+1
        N3=N2+1
        DO 720 M=1,N3
        IF (M.EQ.N2) GO TO 730
        IF (M.EQ.N3) GO TO 740
C
C      ROUTE CATCH BASIN FLOWS TO DOWNSTREAM END OF SEWER
        NC=NUMCS(M,L)
        TIMFLO=DCS(M,L)/SEWV(L)/60.
        DO 745 J=1,NCB
        IF (NUMCB(J).NE.NC) GO TO 745
        JSUB=J
        GO TO 748
745  CONTINUE
748  DO 750 IT=1,NINT
        TIME(IT)=TCL(IT)+TIMFLO
750  FLOW(IT)=QGOUT(JSUB,IT)
        GO TO 760
C
C      ROUTE ROOF RUNOFF TO DOWNSTREAM END OF SEWER
730  TIMFLS(L)=SEWL(L)/SEWV(L)/60.
        IF (NROOFS(L).EQ.0) GO TO 720
        NMARK=0
        TMARKB=-(TRLAG+TINT/2.0)
        IMARKB=0
        DO 770 IT=1,NINT
        IF (NMARK.EQ.1) GO TO 780
        TMARKB=TMARKB+TINT
        IMARKB=IMARKB+1
        IF (TMARKB.GT.0.0) GO TO 790
        QDEL=0.0
        GO TO 800
790  NMARK=1
        NLEN=(TIMFLS(L)-TMARKB)/TINT
        TMARKB=TIMFLS(L)-TMARKB-NLEN*TINT
780  IDASH=IT-IMARKB+1
        QDEL=TMARKB/TIMFLS(L)*QROOF(L,IDASH)
        IF (NLEN.EQ.0) GO TO 810

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      DO 820 NL=1,NLEN
      IF (IDASH-NL.EQ.0) GO TO 800
820  QDEL=QDEL+TINT/TIMFLS(L)*QROOF(L,IDASH-NL)
810  IDASH1=IDASH-NLEN-1
      IF (IDASH1.EQ.0) GO TO 800
      QDEL=QDEL+TMARKE/TIMFLS(L)*QROOF(L,IDASH1)
800  QDOM(L,IT)=QDOM(L,IT)+QDEL
770  CONTINUE
      GO TO 720

C
C      ROUTE UPSTREAM FLOWS TO DOWNSTREAM END OF SEWER
740  IF (ISTORE.NE.4) GO TO 825
      DO 826 NST=1,NSTORL
      IF (NUMUPM(L).NE.LOC(NST)) GO TO 826
      WRITE (6,930) NUMSL(L),(QUPM(L,IT),IT=1,NINT)
      LMARK=L

C
C      ROUTE SEWER FLOWS THROUGH HOLDING RESERVOIR STORAGE
      CALL STOPAG(ISTORE,DSTOR,QSTOUT,QSTO,NINT,TINT,LOC,
1RAIN,QSINRF,QROOF,NCB,NUMCB,QGOUT,QUPM,NROOFS,NSL,NST,LMARK)
      GO TO 825
826  CONTINUE
825  DO 830 IT=1,NINT
      TIME(IT)=IT*TINT+TIMFLS(L)
830  FLGW(IT)=QUPM(L,IT)

C
C      DETERMINE HYDROGRAPH COORDS AT DOWNSTREAM END OF SEWER
760  IT=1
      DO 900 IT1=1,NINT
      TCL1=IT1*TINT
      IF (TCL1.LT.TIME(1)) GO TO 910
      DIFFT=TCL1-TIME(IT)
      IF (IT.EQ.NINT) FLOW(IT+1)=FLOW(IT)
      QDEL=FLOW(IT)+DIFFT/TINT*(FLOW(IT+1)-FLOW(IT))
      IT=IT+1
      GO TO 920
910  QDEL=0.0
920  QDOM(L,IT1)=QDOM(L,IT1)+QDEL
900  CONTINUE
720  CONTINUE

C
C      SUM STORMWATER FLOWS AND SANITARY FLOWS(IF ANY)
      DO 925 IT=1,NINT
925  QSEW(L,IT)=QDOM(L,IT)+QSAN(L)
      WRITE (6,930) NUMSL(L),(QSEW(L,IT),IT=1,NINT)
930  FORMAT (/ ,5X,15,5X,12F8.3,15(/ ,15X,12F8.3))

C
C      DOWNSTREAM FLOWS BECOME UPSTREAM INFLOWS FOR NEXT SEWER
      DO 1020 L1=1,NSL
      IF (NUMUPM(L1).NE.NUMDOM(L)) GO TO 1020
      DO 1030 IT=1,NINT
1030  QUPM(L1,IT)=QUPM(L1,IT)+QDOM(L,IT)
      GO TO 710
1020  CONTINUE
710  CONTINUE
640  CONTINUE
      RETURN
      END

```

```

SUBROUTINE STORAG(ISTORE,DSTOR,QSTOUT,QSTO,NINT,TINT,LOC,
1RAIN,QSINRF,QROOF,NCB,NUMCB,QGOUT,QUPM,NROOFS,NSL,NST,LMARK)
C
C     STORAGE ROUTING SUBMODEL
C     A SUBPROGRAM FOR ROUTING FLOWS THROUGH STORAGE
C
C     DIMENSION QSTO(10),RAIN(161),QSINRF(161),QROOF(56,161),LOC(10),
1INUMCB(129),QGOUT(129,161),QUPM(56,161),NROOFS(56),QA(161),
2QOUT(161)
C
C     IF (ISTORE.EQ.2) GO TO 200
C     IF (ISTORE.EQ.3) GO TO 300
C     IF (ISTORE.EQ.4) GO TO 400
C
C     ROOF PONDING
C     SUMR=0.0
C     DO 110 IT=1,NINT
C     SUMR=SUMR+RAIN(IT)*TINT/60.
C     IF (SUMR.GT.DSTOR) GO TO 130
C     QSINRF(IT)=0.0
C     DO 120 L=1,NSL
120  QROOF(L,IT)=0.0
C     GO TO 110
C     130 TOVER=(IT-1)*TINT
C     WRITE (6,140) TOVER
C     140 FORMAT (//,10X,'ROOF OVERFLOWS AFTER',F8.2,' MINS',//)
C     GO TO 150
C     110 CONTINUE
C     150 RETURN
C
C     SITE STORAGE OF ROOF RUNOFF
C     200 WRITE (6,210)
C     210 FORMAT (//,5X,'ROOFS',5X,'VOLST',/)
C     DO 220 IT=1,NINT
C     220 QA(IT)=QSINRF(IT)
C     QB=QSTOUT
C     GO TO 500
C     230 DO 240 IT=1,NINT
C     QSINRF(IT)=QOUT(IT)
C     DO 250 L=1,NSL
C     250 QROOF(L,IT)=QSINRF(IT)*NROOFS(L)
C     240 CONTINUE
C     WRITE (6,260) VOLST
C     260 FORMAT (10X,F10.2)
C     RETURN
C
C     CATCH BASIN STORAGE
C     300 WRITE (6,310)
C     310 FORMAT (//,5X,'NUMCB',5X,'VOLST',/)
C     J=0
C     320 J=J+1
C     IF (J.GT.NCB) GO TO 360
C     DO 330 IT=1,NINT
C     330 QA(IT)=QGOUT(J,IT)
C     QB=QSTOUT
C     GO TO 500
C     340 DO 345 IT=1,NINT
C     345 QGOUT(J,IT)=QOUT(IT)
C     WRITE (6,350) NUMCB(J),VOLST
C     350 FORMAT (5X,I5,F10.2)

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      GO TO 320
360 RETURN
C
C      HOLDING RESERVOIR STORAGE (LINEAR OUTFLOW)
400 WRITE (6,410)
410 FORMAT (//,7X,'LOC',5X,'VOLST')
      DO 420 IT=1,NINT
420 QA(IT)=QUPM(LMARK,IT)
      QB=QSTO(NST)
      GO TO 500
430 DO 440 IT=1,NINT
440 QUPM(LMARK,IT)=QOUT(IT)
      WRITE (6,350) LOC(NST),VOLST
      RETURN
C
C      ROUTE FLOWS AND CALCULATE STORAGE REQUIREMENTS
500 VOLST=0.0
      VOLLF=0.0
      IRISE=0
      DO 510 IT=1,NINT
      IF (QA(IT).LT.QB.AND.IRISE.EQ.0) GO TO 515
      IF (QA(IT).LT.QB.AND.IRISE.EQ.1) GO TO 520
      IRISE=1
      VOLST=VOLST+(QA(IT)-QB)
      GO TO 530
520 VOLLF=VOLLF+(QB-QA(IT))
      IF (VOLLF.GT.VOLST) GO TO 515
530 QOUT(IT)=QB
      GO TO 510
515 QOUT(IT)=QA(IT)
510 CONTINUE
      VOLST=VOLST*60.0*TINT
      IF (ISTORE.EQ.2) GO TO 230
      IF (ISTORE.EQ.3) GO TO 340
      IF (ISTORE.EQ.4) GO TO 430
      RETURN
      END

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The effect of storage on storm
and combined sewers / Henry,
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